THE ARUP JOURNAL

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Contents

Errata
Arup Journal March 1969
P. 12 Fig. 3 (b) was printed upside down
P. 14 Fig. 7. The caption should have read
Wind pressure envelope on tower
P. 15 Fig. 8 The caption should have read
Rotation of floor below fourth floor level

Front Cover: Coalmine winding gear (Photo: National Coal Board).
Back Cover: The Crab Nebula in Taurus. (One of the most powerful radio sources in our galaxy).
(Photo: Mount Wilson and Mount Palomar Observatories).

Lletty Shenkin tips, Cwmbach

Brian Corbett

At Aberfan on 21 October 1966, about 140,000 tons of material slipped from Tip 7 and engulfed part of the village. 144 people were killed, of whom 116 were children. One of the questions which had to be asked, following the disaster, was whether this was a unique occurrence or whether similar slides had occurred before. The National Coal Board has identified some 650 individual tips within its ownership in South Wales and there are probably as many again in private ownership. Some of these have slipped and recent investigations have shown that others are potentially unstable.

As part of the development in the Cynon Valley near Aberdare it is proposed to build some 360 houses at Rose Row, Cwmbach. The architects for the scheme are the Welsh Office, Cardiff and the client is the Urban District Council of Aberdare. The site is on the east side of the valley and is on a gentle slope rising towards the Merthyr Mountain. The site is dominated by two waste tips forming part of the spoil from the workings at the former upper Lletty Shenkin Pit.

The tips—general

An aerial photograph of the tips looking in an easterly direction is shown in Fig. 2. In the early days of the colliery the spoil was tipped around the pit-head and the lower tip was formed during the period from 1868 to 1900. Subsequently the upper tip was mainly formed during the period 1900 to 1914. There was little further output from the colliery until it was closed in 1927.

Mining history

The Lletty Shenkin Pit was opened in 1843. Since that time eight seams have been worked under or near the site at depths between 400 and 800 ft. (122 m and 244 m). There has been no extraction for nearly 50 years, except for the deepest seam, The Gellideg, from which coal was removed at an adjacent colliery up to the western edge of the site in 1958. We understand from the National Coal Board that no further workings beneath the site are contemplated.

Fig. 1
Longitudinal section
(Drawn by: Don Jordan)
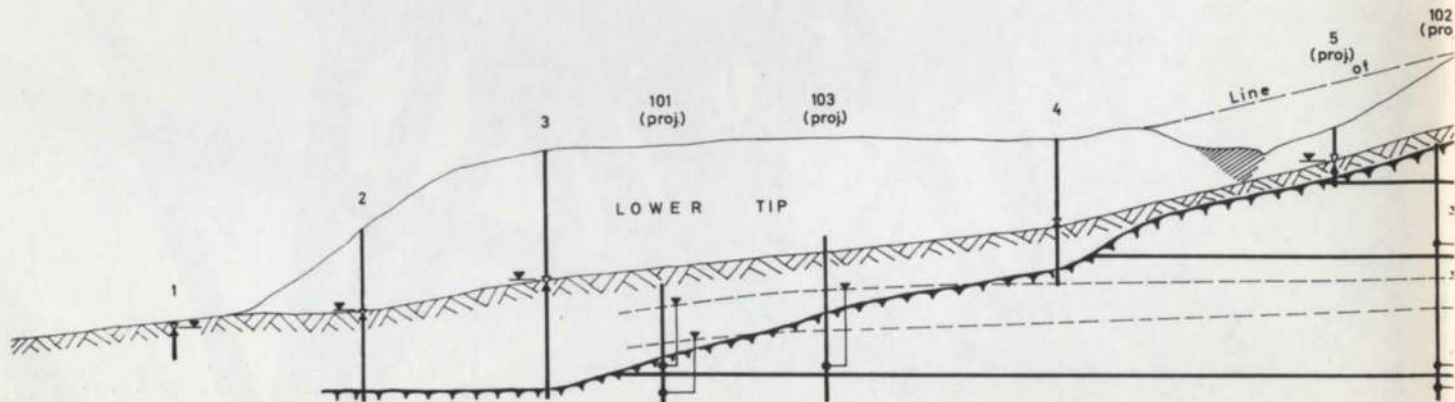
In addition to mining from the upper Lletty Shenkin shaft, drift workings on the hillside took place up to about 1875. Mouths of old adits can be found on the hillside. The Rhondda No. 2 and possibly the seam immediately beneath it have been worked in this manner.

Problems presented by spoil tips

The main growth in industrial development in the Welsh Valleys was in the second half of the 19th century. The valleys are relatively narrow and they quickly became congested with houses, pit head equipment, transport facilities and colliery waste. The physical nature of most of the valleys dictates the arrangement of this development. Houses were usually built close to the pit head, and consequently soon became dominated by colliery equipment and by mounds of colliery shale waste. With an acute shortage of level land, both houses and waste tips crept up the steep valley sides.

In the past, colliery waste tips have often been constructed with little regard for long-term stability. In particular, the practice of dumping on a hillside, combined with a disregard of the soil, rock and ground water conditions has led to many cases of movement and failure. The possible modes of failure are as follows:

Rotational Slip

At all locations where the ground is not level, there are forces acting which tend to cause movement of soil from high points to low points. The most important of such forces is the component of gravity that acts in the direction of probable motion. Also important, but not so well recognized, is the force of seeping water. These forces cause shearing stresses throughout the soil, and a mass movement occurs unless the shearing resistance on every possible surface through the mass is of sufficient magnitude to withstand these shearing stresses. When a movement occurs, it is generally of a rotational nature.

Liquefaction

This can occur in a heap of loose sand or in an uncompacted tip of mine rubbish. If the lower part of the tip contains water filling the spaces between the particles, and if a sudden load or shock is applied (such as a slip in a tip in the form of a rotational slide), the water then separates the particles and the whole saturated body behaves as if it were a liquid.

Fig. 2
Aerial photograph of Lletty Shenkin tips
(Photo: H. Tempest (Cardiff) Ltd.)

Flow-slide

If liquefaction occurs, in such a heap, on a slope, the mass will rush down the slope as if it were a liquid, although it is actually a mass of wet solids. When it stops it immediately reverts to a relatively dry heap.

Mud-run

This term is usually applied to incidents of common occurrence in mountainous districts. Torrents of water rushing down a mountain-side collect, and carry with them, all available loose material in their path—in a 'run of mud'.
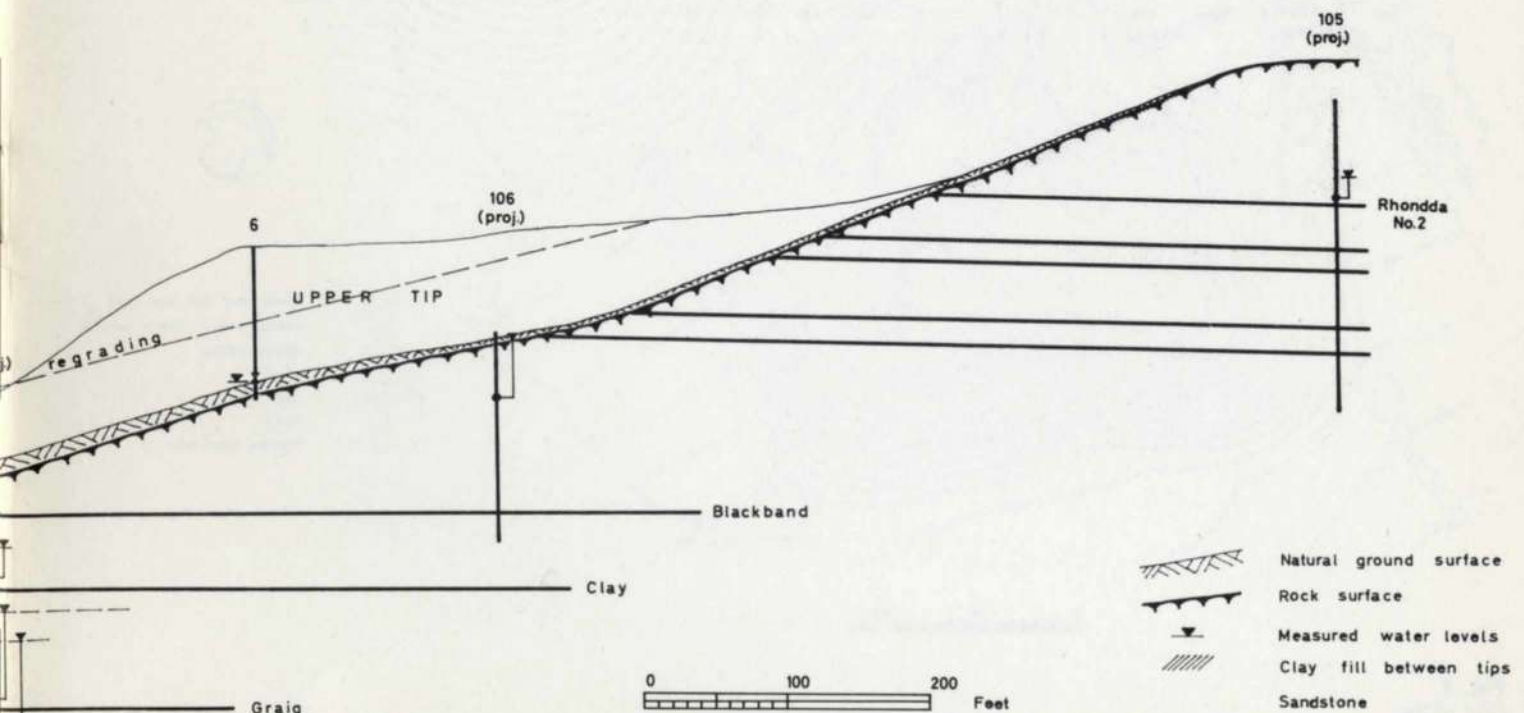
Lletty Shenkin tips

A contour plan of the tips is shown in Fig. 3 and a longitudinal section in Fig. 1. At this site, two possible modes of tip failure need to be con-sidered. The first mode is a rotational slide; this could then provoke liquefaction followed by the second mode, a flow-slide. At the time the investigation was planned, it was considered that a failure of either type in the lower tip could have serious consequences for the housing development. A flow-slide from the upper tip would also be dangerous, but a rotational slip would probably not be, unless it was such an extensive movement that it disturbed the tip below.

Flow-slides behave like liquids and can travel far and fast with little or no warning. They are, therefore, much more dangerous than rotational slips.

To enable the possibility of such failures to be

105
(proj.)

106
(proj.)

6

UPPER TIP

regrading

Rhondda No.2

Blackband

Clay

Graig

| | Natural ground surface |
| Rock surface |
| Measured water levels |
| Clay fill between tips |
| Sandstone |

0   100   200
Feet

assessed, the initial site investigation prog-
ramme of four shell and auger boreholes and
three diamond core drillholes was planned. As
the investigation progressed, it became clear
that these would give insufficient information,
and so two more boreholes and two more
drillholes were recommended.

Site investigation

To investigate the stability of the upper and
lower tips, a programme of boreholes and
drillholes was put in hand. This programme
was to determine the properties of the material
in the tips, the nature of the drift and the
properties of the bedrock. The water pressures
were measured in the tips, below them and
also at different horizons in the rock.

Geology

A longitudinal section giving the geology is
shown in Fig. 1.

Drift

Glacial deposits of boulder clay cover the
valley floor, and have a thickness of about
60 ft. (18 m) beneath the western part of the
lower tip, and run out on the hillside on a line
passing beneath the upper tip.

On the hillside overlying much of the boulder
clay is a layer of head, which consists largely
of weathered coal measures series rock which
has been moved down the hillside by creep
and solifluction. The head is 1 ft. to 2 ft.
(0.3 to 0.6 m) thick beneath the upper tip and
increases to 5 ft. to 10 ft. (1.5 to 3 m) thick
beneath the lower tip.

Solid

Bedrock is of the Coal Measures series and the following succession was found:

Level ft. N.D.	Thickness	Strata
430 (131 m)	–	Graig Coal Seam.
–	80 ft. (24 m)	Siltstones and mudstones
510 (155 m)	–	Clay Coal Seam – the Upper Cwmgorse Marine Band immediately above this seam is the boundary between the Upper and Middle Coal Measures.
–	60 ft. (18 m)	Llynfi Beds—siltstone.
570 (174 m)	–	Blackband Coal Seam.
–	210 ft. (64 m)	Llynfi Beds—alternations of mudstone and siltstone containing four coal seams.
780 (238 m)	–	Rhondda No. 2 Coal Seam
880 (268 m)	100 ft. (30 m)	Pennant Sandstone

The strata dip at about 1 in 12 to the south-
east. A fault is shown on the geological map,
crossing the upper part of the upper tip, from
north-west to south-east.

Water pressures

The water pressures measured are also shown
in Fig. 1. These show that the ground water
level is below the level of both tips. It is worth
noting that there is generally a difference in
pressure above and below a coal seam—the
pressure is higher immediately above the
seam.

Lower tip

The material constituting this tip appears to
be almost entirely burnt. That is to say when it
was originally excavated and tipped it con-
sisted of broken pieces of mudstone, siltstone
and coal, but since then, due either to
spontaneous combustion or to dumping of
burning boiler ash on it, the coal has caught
fire and the heat has baked the siltstone and
mudstone into a material resembling brick.
The material has fused together in places, and
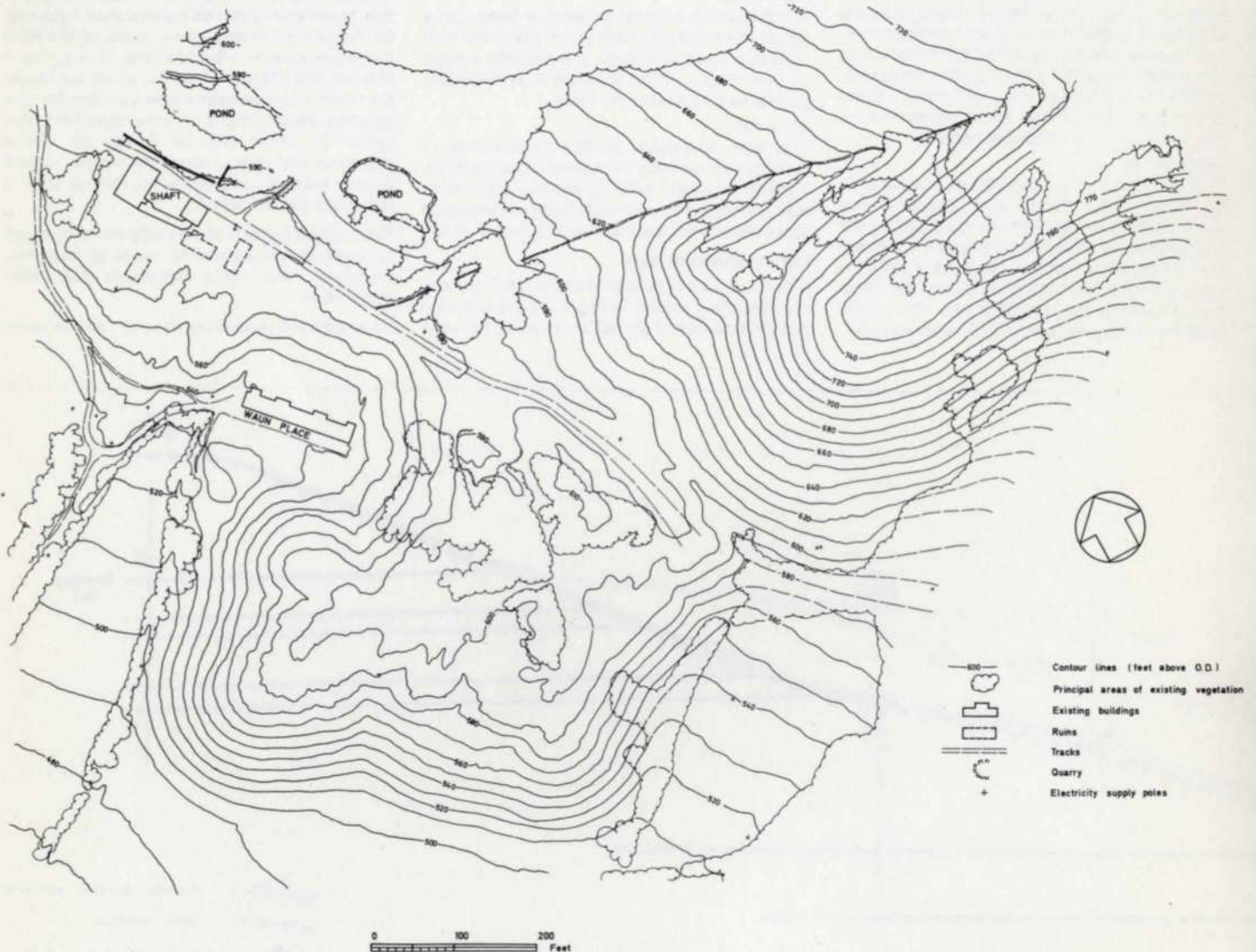seems entirely resistant to any breakdown due

0   100   200 Feet

Contour lines (feet above O.D.)
Principal areas of existing vegetation
Existing buildings
Ruins
Tracks
Quarry
Electricity supply poles

Fig. 3
Plan of tips
(Drawn by: Don Jordan)

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to weathering effects. Beneath the tip, clay layers were found in two of the three boreholes. No water was found in the tip, and the groundwater level coincides with the base of the tip.

With regard to the dense nature and the fusion of the tip material it is considered that a rotational slide of this tip will not occur. If no rotational slide occurs there is no risk of the more dangerous flow-slide. A simplified stability analysis was carried out, considering the bearing capacity of the underlying boulder clay. This gave a high factor of safety against bearing capacity failure.

This tip is covered with well-established vegetation. It requires no remedial measures other than local ones for landscaping purposes.

Upper tip

This tip is composed of a mixture of broken mudstone and siltstone and coal. At the surface, weathering has caused a breakdown to fine particles. The faces stand at steep angles and, although the crown towards the hillside is well covered with grass, shrubs and trees, vegetation has only a tenuous hold on the slopes. In parts, grass has been entirely removed either due to frost action or to excessive rainfall run-off, causing scouring and mud-runs. There are signs of movement in two places at the western end of the tip, seen as bulges in the tip face.

The tip has been formed over a line of springs,

and possibly across a small stream. However, no water was found within the tip and the groundwater level is at, or below, the base of the tip.

Stability analyses show that although the tip is stable at the present time, it is unlikely to remain stable in the long-term—i.e. during, say, the next 50 to 60 years. There are two main reasons for this. Firstly, the tip material breaks down with time due to weathering processes and to local movements. This causes a progressive reduction in its shear strength. Secondly, it is likely that the flow of groundwater from the hillside will increase as further pit closures take place.

Remedial measures

To ensure stability of the upper tip it is necessary to reduce its height so that it has a factor of safety of 1 against long-term failure. A reduction in height of the tip to about 50 ft. (15 m) above the hillside, coupled with a flattening of the side slopes on the southern side, is necessary. These proposals are shown in Fig. 4. This requires the removal of about 73,000 cu. yd. (56,000 m³) of spoil, and its retipping in the area between the two tips, in the existing hollow in the lower tip (which contains Waun Place) and in the two ponds formerly used to supply water at the pit head. Some regrading immediately to the west of the old pit head is suggested. (This involves a further 19,000 cu. yd. (14,500 m³) approximately of material.) In order to ensure that

stability in the future is not imperilled by water seepage beneath the tips, some additional drainage is necessary. The proposal is for a drain along the northern side of the upper tip which will intercept surface water and also water flowing from the sandstone outcrops in the bedrock. The discharge will be fed through the new drainage system for the housing development and discharged into the River Cynon.

The approximate quantities of cut and fill are:

Cut

upper tip	73,400 cu. yd. (56,100 m³)
west of pit head	18,700 cu. yd. (14,300 m³)
total	92,100 cu. yd. (70,400 m³)

Estimated volume after compaction ⅚ × 92,100 = 76,700 cu. yd. (58,600 m³)

Fill

between tips	43,400 cu. yd. (33,200 m³)
Waun Place	27,600 cu. yd. (21,100 m³)
ponds	6,400 cu. yd. (4,900 m³)
total	77,400 cu. yd. (59,200 m³)

Programme

The final report on the investigation was presented to the Urban District Council of Aberdare during April 1969.
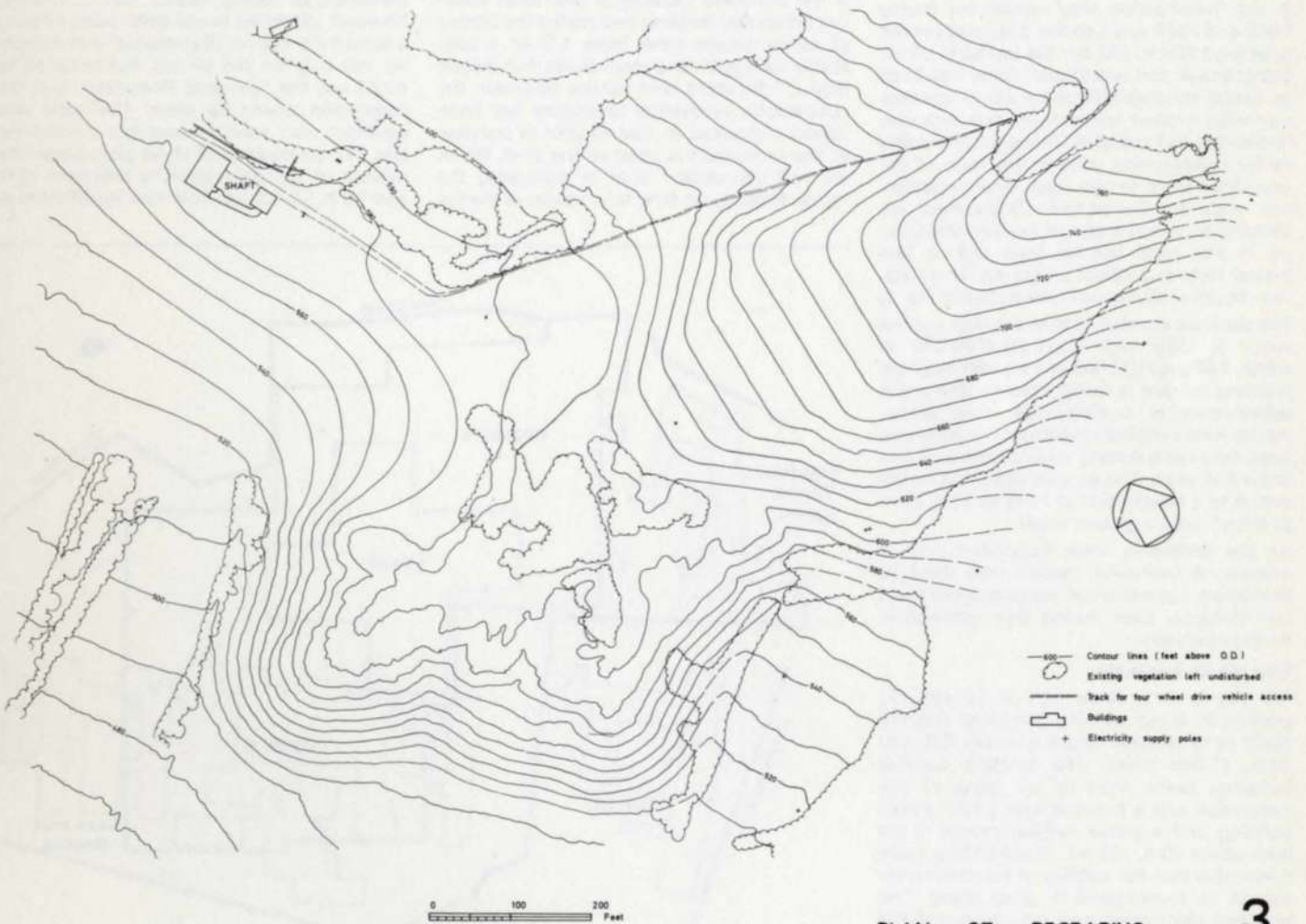

Contour lines (feet above O.D.)
Existing vegetation left undisturbed
Track for four wheel drive vehicle access
Buildings
Electricity supply poles

PLAN OF REGRADING 3

Fig. 4
Plan of tips after regrading
(Drawn by: Don Jordan)

Royal Free Hospital, Hampstead

Roger Findlay, Joe Levy and Jack Waller

General

The new Royal Free Hospital at Hampstead has been under construction since June 1968. The total cost of the building works will be something in excess of £12m.

The hospital is being built off Haverstock Hill behind Hampstead General Hospital and immediately north of Lawn Road Hospital. In the second stage of the project these existing hospitals will be demolished, but for the time being both of them have to be kept in operation. The new hospital has to be built very close to the existing buildings and this has been a major factor in the design.

The new hospital is an 18-storey cruciform tower rising from a podium which varies in height from one to seven storeys. The variation in height of the podium is partly accounted for by the planning arrangements and partly by the steep fall of the ground across the site. The layout is shown in Fig. 1.

Site investigation

A site investigation was carried out during 1965 and 1966 and London Clay was proved to at least 200 ft. (60 m), the top 30 ft. (9 m) being brown and weathered. There had been an earlier investigation of the site in connection with another scheme and this data was fortunately still available. The results from this earlier investigation were in the form of the usual undrained triaxial tests. After consultation with the Geotechnics Department we decided to concentrate, in our investigation, on in situ plate bearing tests with a few triaxial tests as a check on the earlier results. The results of all this work are shown in Fig. 2.

The sulphate content of both the clay and the water is fairly high (335 pts./100,000 in water, 147 pts./100,000 in soil) and sulphate resisting cement is being used for the entire substructure in consequence. Four piezometers were installed in various bore holes and were read continuously over a period of two and a half years. The ground water eventually settled to a steady level at 10 ft. to 15 ft. (3 m to 4.5 m) below ground level.

All the boreholes were back-filled with a mixture of bentonite, cement and sand as precaution against water passing down them and flooding piles during any subsequent boring operation.

Slip circle analysis

The site is on a slope of 1 in 10 and the excavation is cut into the hillside so that the depth of excavation ranges between 6 ft. and 36 ft. (1.8 m-11 m). The existing hospital buildings come right to the brink of the excavation and a convent with a four-storey building and a brittle looking chapel is set back about 50 ft. (15 m). These factors made it essential that the stability of the excavation should be investigated in great detail. The potential slip circles were very large and the forces involved were huge. It was, therefore, essential to maintain an adequate factor of safety, since once a slip began to develop it would be impossible to contain it. Altogether 7,000 slip circles were analysed using computer program numbers OA83, 801 and 802, at a cost of less than £150. By hand, it would have taken over 60 man years!

The slip circle programs were originally written for the Royal Free Hospital, but during development they were also used on the Gateshead Bypass. The programs were based on Bishop's simplified method of slices. They passed through various stages of development as their limitations were discovered. It was soon realized that it was not sufficient to have a constant value for cohesion for short term analysis. As the radii of the circles increased, the factors of safety went down and this was not likely when the strength of the clay increased with depth. As a result OA802 was written. With some sections, we were finding that contours of factors of safety (see Fig. 3), would not close and that as the radius increased, the value of the factors of safety increased in cycles. This was traced to the fact that unless the circle cut the mid ordinate of a slice the whole of the slice was ignored. This worked against us because generally the reduction in overturning moment was considerably less than the decrease in resisting moment due to shortening of the slip circle. The program was amended to cut out this anomaly. These are two examples of improvements that were made. Since we have finished our work, the programs have been rewritten based on Bishop's Rigorous Analysis, OA804, which gives far better factors of safety. It would be very interesting one day to write the life story of a set of programs such as this.

The soil parameters assumed are for long term, cohesion $C = 250$ lb/ft.^{2} (11.97 kN/m^2) and angle of internal friction $ø = 20°$ and for the short term c varies according to depth, the values being taken from the plate loading tests.

If the complete excavation had been taken out before any building had started the factors of safety would have been 1.0 or a little above, using the long term values, but greater than 2.0 for short term values. However, the contractor's excavation procedure has been carefully checked so that as little as possible of the excavation is open at any time. When the raft foundation only is complete, the factor of safety on long term values is always over 1.3. This value is not only important during construction, but should there be a disastrous fire, for example, and the frame had to be demolished down to the foundations, the basements would still be stable. With the full dead load of the building on the foundation, the factor of safety is always greater than 2.0. In all cases it was not sufficient to add in just the vertical load from the structure, it was necessary to feed in manually into the critical circles, both the horizontal reaction provided by the rafts, and the strength of the rafts themselves.

At the time of writing, it is intended that the critical circles be re-analyzed using Bishop's Rigorous Method, to check whether we have underestimated the stability.

Retaining walls

In the south and south-western areas, the retaining walls cantilever off the rafts; the excavation being open ended and there being no effective prop. Using the parameter $c = 250$ lb/ft^2 and $ø = 20°$ and with the water table about 15 ft. (4.6 m) below ground level, it was found that soil pressures approximated to 60h, where h is the depth of soil retained, and this figure was adopted throughout. At the deepest part, the walls are 3 ft. (0.9 m) thick and reinforced with 1.6 in. (40 mm) bars at 6 in. (150 mm) centres. Where walls are to be constructed in open cut and can be drained, they have been designed for $p = 40h$.

At the time of tender the contractor was provided with a scheme of construction in which the temporary works consisted of a bored pile wall of 3 ft. (0.9 m) diameter bored piles at 3 ft. 6 in. (1.07 m) centres, supported by conventional raking struts. The contractor, however, proposed to use sheet piles, offering a substantial saving. His proposal was accepted, not only for the saving, but because he could use the Taywood Pilemaster, and the installation would be silent. The latter was essential with existing hospitals around the site. The penetration of sheet piles below the bottom of the excavation is between 10 ft. and 15 ft. (3-4.6 m). This was insufficient to
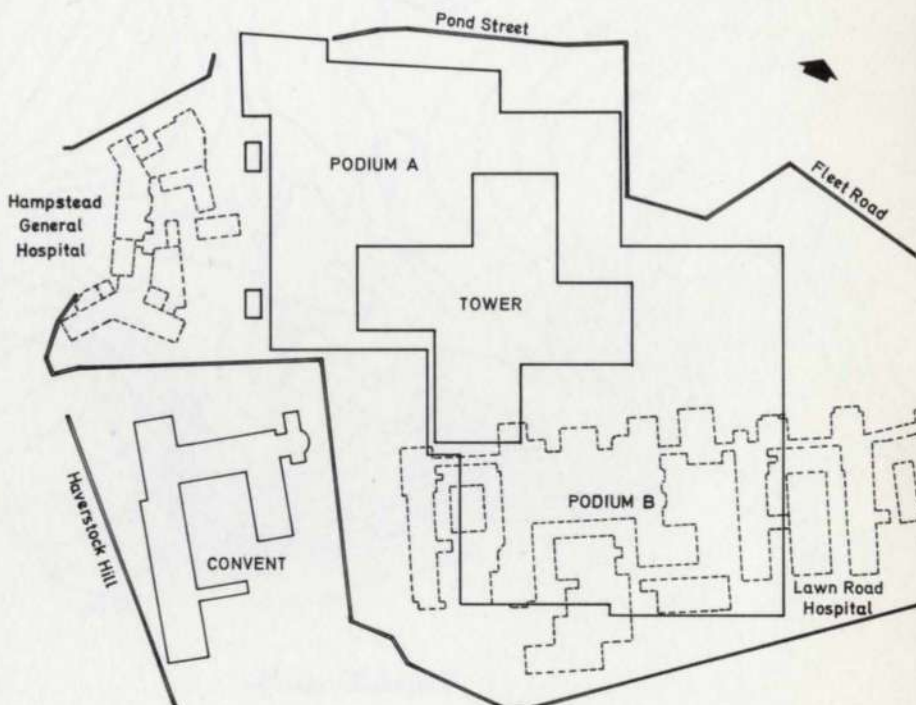

Fig. 1
Hospital site plan
(Drawn by: Marjorie Bishop)

cut off slips in the deep excavation adjacent to the convent where the future building loads are low and the bored piles were retained, giving penetrations of 20 ft. (6.1 m). The contractor has subsequently experimented with a scheme using ground anchors which are bored through the sheet piles and into the ground behind. This idea offered considerable advantages in that the excavation would not be obstructed by the raking struts, but difficulties were encountered, and the idea has had to be dropped for this particular job.

Tower foundations

The weight of the tower is approximately 2½ tons/sq. ft. (268 kN/m²) over its plan area. The average weight of soil removed from the excavation is approximately 1¼ tons/sq. ft. (134 kN/m²), leaving a net increase of load on the ground of 1¼ tons/sq.ft. (134 kN/m²). This loading varies a bit, since the depth of excavation is not constant. At this loading, a raft is possible and this form of foundation has been selected, not only because it is marginally cheaper than piles, but also because it provides something against which to strut the retaining walls on the south side of the site. This is a particularly important consideration since the new retaining wall is, in places, hard against the existing Lawn Road Hospital.

A 4 in. (100 mm) settlement has been estimated in the centre of the raft at 100 years.

Analysis of tower raft

The raft under the tower is 7 ft. (2.13 m) thick. It contains 18,000 cu. yd. (13,760 m³) of concrete and 1,400 tons (1,422 tonnes) of steel. Three matters had to be looked at in considerable detail.

a) Assessment of the distribution of contact pressure under the raft.

b) Design of the raft on the basis of the contact pressures and the applied column and wall loads.

c) Assessment of the minimum reinforcement required to resist shrinkage stresses.
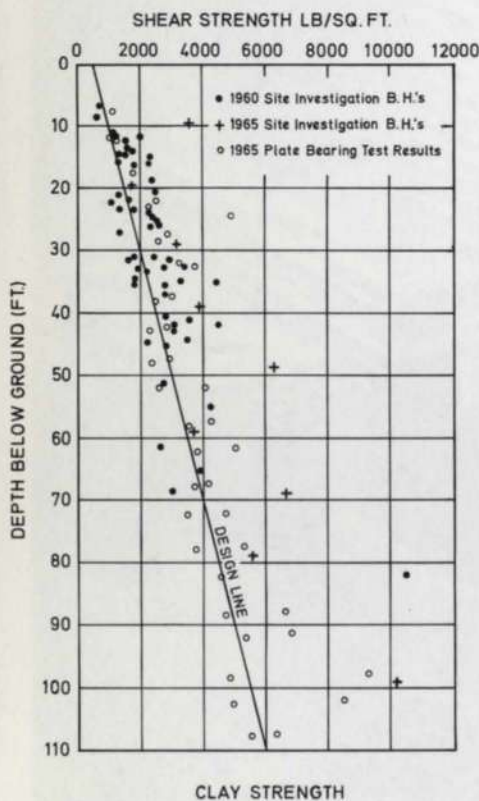
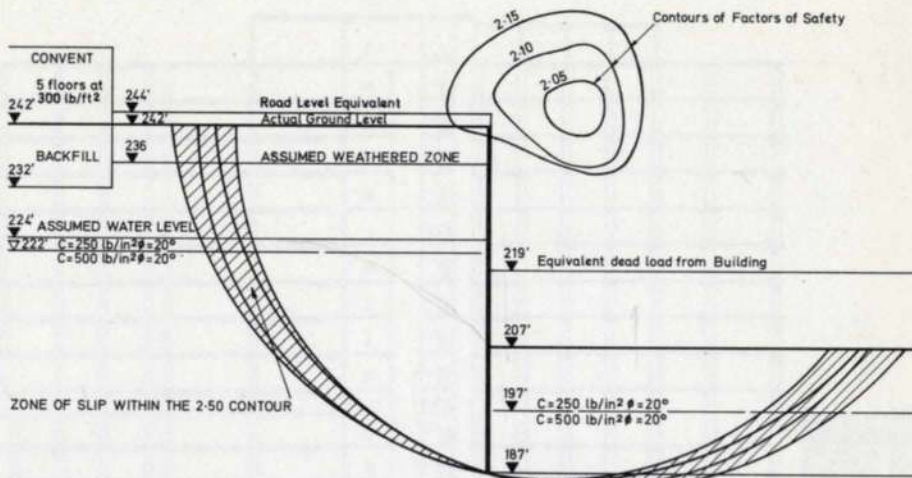Several theories exist on the contact pressure

Fig. 3
Typical slip circle analysis
(Drawn by : Marjorie Bishop)

distribution under a raft (Baker's Soil-Line Method, Laing Barden, etc.). The method used was a modified version of Baker's, the difference being that the soil settlement profile was obtained by Boussinesq stress functions using the computer program developed to calculate elastic settlements. The essence of the method is to start off with an assumed U.D.L. applied by the raft to the soil and to reduce the incompatibility between the raft and the ground line by a process of successive corrections (Figs. 4, 5 and 6). The method was refined by considering the raft as a grid framework supported by springs at the node points; the spring stiffness being a function of the soil settlement under the node load. The column loads were converted to equivalent node loads (Fig. 7). This involved using the computer with the grid framework program and a specially written settlement program. A complication arose from the fact that the raft structure alters the distribution of the original column loads at their point of contact between the raft and the ground. This redistribution is a function of the raft stiffness and the soil stiffness. Also the spring stiffnesses were affected by the fineness of grid and the effect of U.D. versus point loading. An iterative process was used until the difference between two successive sets of spring stiffnesses was negligible, the iteration being done by inspection.

Having obtained the contact pressures and the overall deflected shape of the raft, the reinforcement was calculated for overall bending of the raft, and for local moments and shears at columns and walls. The raft was divided into suitable column strips and slabs for the purpose of resisting local moments and shears. Problem areas indicated by the computer were given special attention.

In view of the enormous volume of concrete involved, and the agreed size of a day's pour (200 cu. yd.: 153 m³) it was necessary to investigate the effect of shrinkage stresses in the raft. Two effects were considered, (1) the raft shrinking uniformly throughout the entire thickness, and (2) the outside surfaces of the raft drying at a faster rate than the inside and setting up differential stresses in the concrete. In addition, the reinforcement calculated in these two cases had to be checked against a minimum percentage figure (say 0.25%) conforming to good practice. The final percentage of reinforcement worked out at 0.5%.

Podium foundations

The podium is in two parts; Podium A to the west of the tower and Podium B to the east. Over a considerable area of Podium A a substantial excavation has to be taken out,

Fig. 2
Clay strength
(Drawn by : Marjorie Bishop)
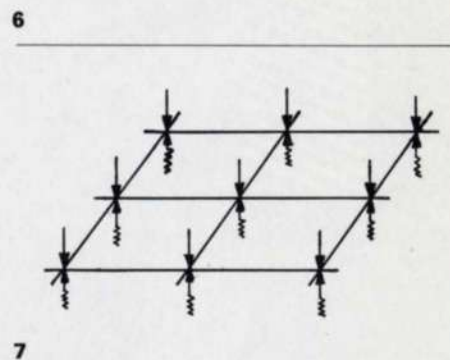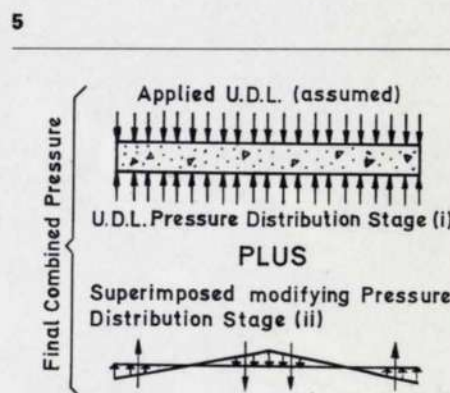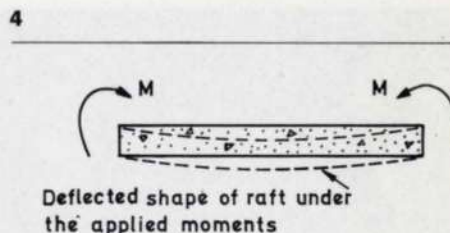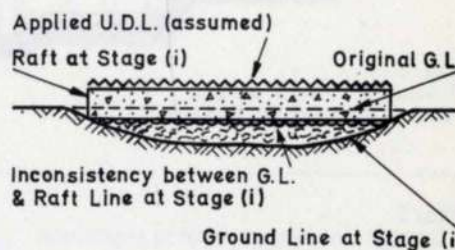
Figs. 4 to 7
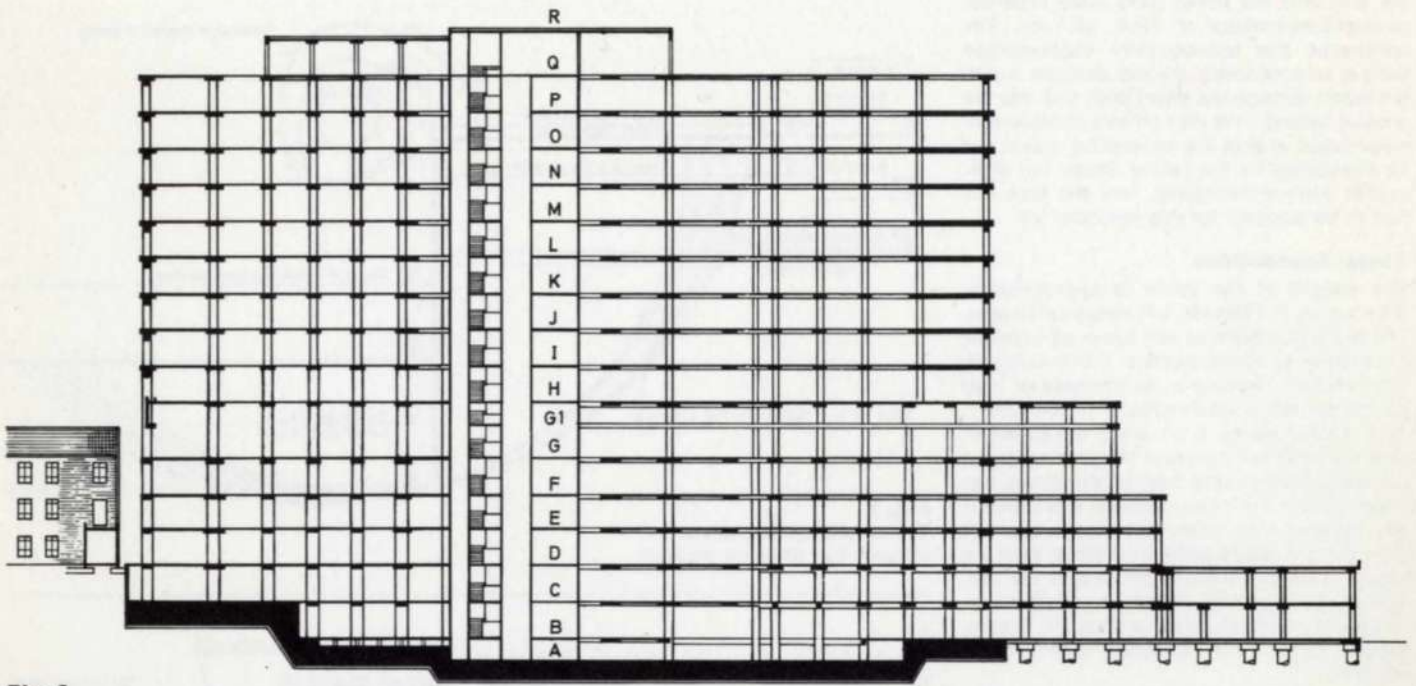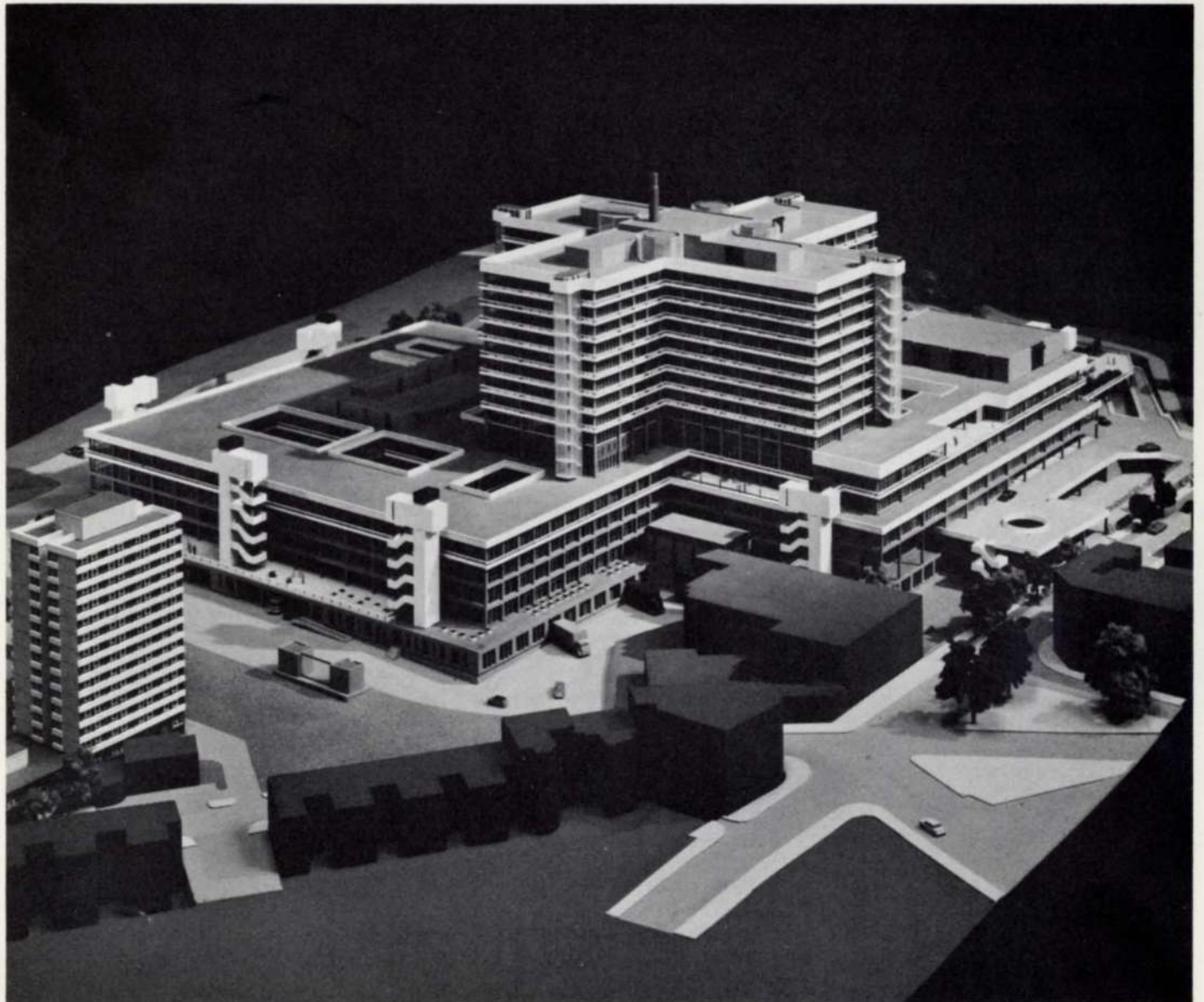Sequence of raft design
(Drawn by : Marjorie Bishop)

Fig. 8
Typical north-south section
(Drawn by: Marjorie Bishop)

Fig. 9
Proposed development looking south-west
(Photo: John Maltby Ltd)

resulting in a net reduction of load on the ground. The clay in this area is, therefore, expected to heave due to the release of pressure. It was originally planned to support the podium on piles and leave a heave space below the lowest floor level in the same manner as at B.P. House. However, the deeper part of the podium is now supported on a raft. The change came about for three reasons. First of all, a more detailed analysis showed that the heave space was only marginally necessary and then only in one corner, secondly, it was necessary to provide something against which the retaining walls on the south and west sides could be strutted in both the temporary and final conditions, and thirdly we wanted to use the weight of the building to increase the factor of safety against rotational slip. Podium B foundations are much less complicated since the excavation is small. It is founded on large diameter bored piles throughout.

Drainage

Under most of the building there is a 3 ft. (0.9 m) void below the lowest floor level to accommodate services. Where there is a raft this void occurs between the top of the raft and the first occupied floor, which is supported off the raft on piers at about 8 ft. (2.44 m) centres. Where there are piles, the void is formed in effect between the site blinding and the lowest structural slab. In both cases the opportunity presented by this void

has been taken to use it for drainage of any groundwater that may appear. In the piled areas, any groundwater will be collected in a 6 in. (150 mm) layer of no-fines concrete laid on top of the blinding and between the piles, similar to Tower Hill.(1)

Where possible the retaining walls have drains behind them. Where the retaining walls are formed in front of bored or sheet piles and drainage behind is not possible, a drained cavity will be constructed in front of the wall.

Superstructure

The column grid in both the tower and Podium A is approximately 32 ft. by 16 ft. (9.75 m x 4.9 m). The floor structure is a series of continuous beams spanning 32 ft. (9.75 m) supporting a floor slab which spans 16 ft. In order to obtain a minimum depth structure the beams are wide and shallow, being only 16 in. (400 mm) deep. The floor slabs are generally 5 in. (130 mm) deep. The need for a minimum depth structure was dictated both by planning requirements, which impose overall height limitations on the building, and also by the large quantity of services which have to be allowed for beneath each floor slab. A typical north-south section is shown in Fig. 8.

In Podium B the column grid is nearly square and in this case 14 in. (350 mm) and 16 in. (400 mm) deep coffered slabs are used; the difference in thickness being in the topping.

A great deal of thought has been given to the problem of cutting future holes for services. In the case of the tower and Podium A, holes can be cut almost anywhere in the 5 in. slabs between beams, and in the case of Podium B, holes can be cut through the tops of almost any coffer. This gives a fairly high degree of flexibility for provision of future services and is the maximum that can be provided without significantly increasing the cost of the structure.
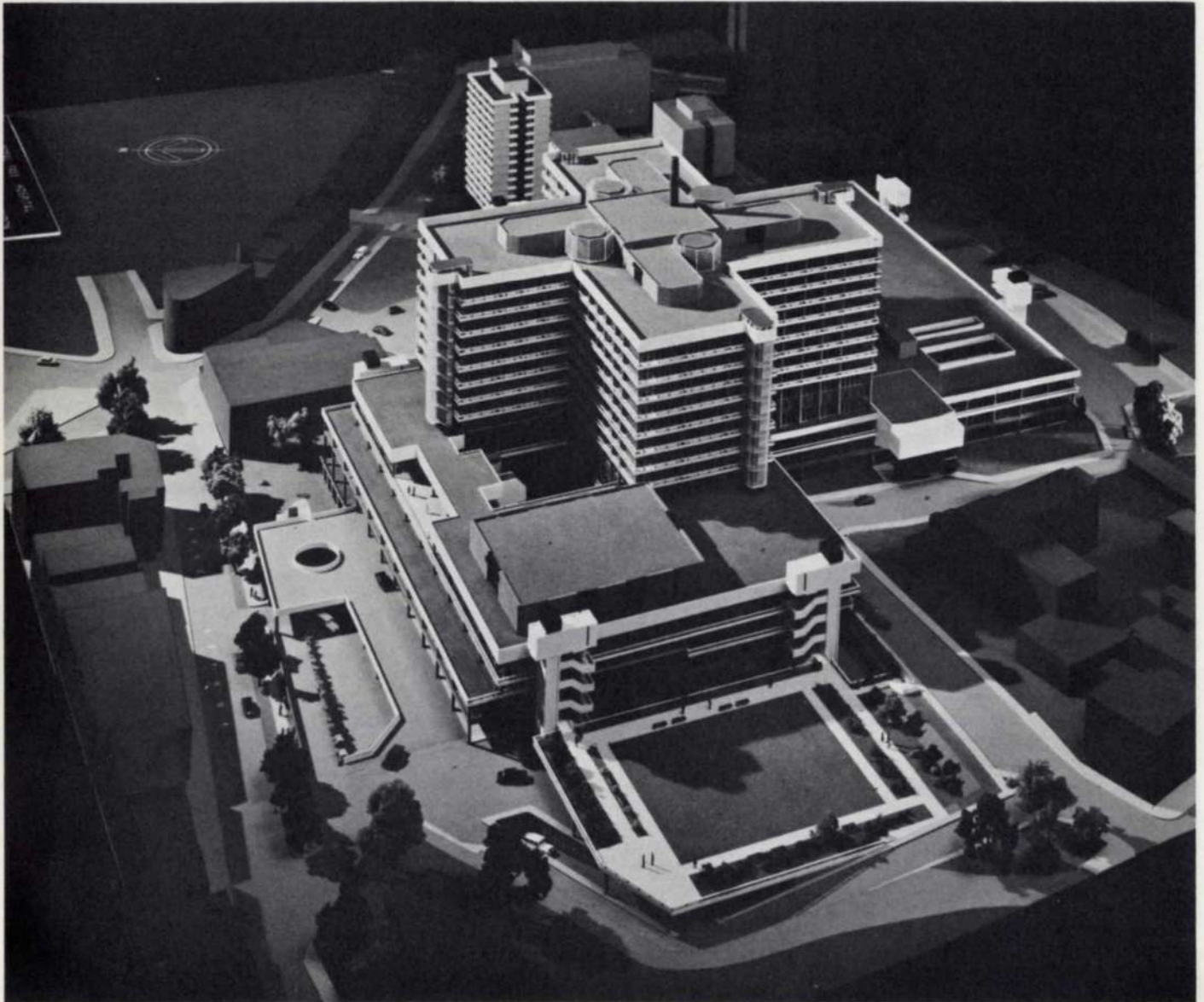
Reference:

(1) HENRY, T. Tower Hill development. *The Arup Journal*, **2** (7), pp. 4-7, 1967.

Fig. 10
Proposed development: view from Haverstock Hill
(Photo: John Maltby Ltd.)

A fabricator's view of timber engineering

Bill FitzGerald

Note

At the present time the timber industry has not agreed on the recommended sizes for metric conversions. As it would, therefore, be confusing to give S.I. equivalents in this article, no metric conversions have been made.

Foreword

Practically everyone at some time or another has had to repair a timber gate, post or fence which has been damaged by an attack of beetle or fungus. This not uncommon occurrence tends to instil a prejudice against timber and raise a mental barrier which is extremely difficult to overcome. For this reason structural design has to face a bigger challenge in timber than with other media.

When an architect asks for an alternative price for his project in timber to compare with the price of the job in steel the former must be more economical in initial cost or it is in danger of being immediately abandoned without due consideration being given to long term advantages. It is significant that an architect, having once tried timber on a project, and, in particular, laminated timber, frequently seeks to repeat the experiment.

Although timber is one of the oldest materials it is also one of the most undeveloped. At the risk of offending some worthy people I would like to state some of the causes.

1. The professions

a) The design approach often lacks the practical know-how.

b) There is a tendency in designing to specify high grade timber which is not readily available and then at very high prices.

c) Many designers attempt to use timber in forms of construction which are only suitable for steel or concrete.

2. The fabricator's work shop

Often a lack of ambition and imagination is displayed in the reluctance to try different forms of fabrication. All too often, the joiner, who learned his trade 30 years ago, dictates the approach to jointing, etc.

3. Research and development

This is way behind design practice in many ways. Although certain groups are doing extremely useful work others are spending time on projects which are regarded by industry as non-starters.

One well-known association offers its service for guidance to professional offices designing timber projects. Theoretical advice, with little practical backing, is offered which can create difficulties for designer and fabricator alike.

4. C.P. 112 : 1967

Although the 1967 edition of this Code of Practice shows some improvement on the previous issue it is still far from being a satisfactory document.

The design of compression members, for example, involves the use of factors which can be arrived at only by the application of involved formulae. Even to experienced timber designers the work involved in strut design could be formidable. It is likely to deter the engineer, making his first acquaintance with the code, from persevering with his experiment in timber design.

The depth to width ratios applied to glued laminated beams are unrealistic and not based on acceptable criteria. The recommended ratios make for uneconomical design of beams unless more involved calculations are undertaken.

Timber availability

The principal species are set out below.

Type	Source
Redwood (Yellow Deal)	Europe, Scandinavia
Whitewood	Scandinavia, Russia
Douglas Fir	British Columbia
Western Hemlock	British Columbia

European timber

Available in sizes up to 11 in., the usual maximum being 9 in. Board thickness available ¾ in., ⅞ in., 1 in., 1¼ in., 1½ in., and 2 in. in lengths up to 16 ft.

Douglas fir

Logs or baulks are very large and can be 24 in. x 24 in. x 60 ft. long. Hence sawing is not very accurate, i.e. 1 in., 2 in., 2½ in. and 3 in. boards.

Properties and uses

Hemlock

May be surfaced to Canadian Lumber Standards. It is planed all round and because of this it is extremely popular with builders. However, the commercial use of hemlock in GluLam construction is not acceptable to some fabricators on account of its subsequent splitting when kiln dried.

Douglas fir

This is popular with design engineers because the Code of Practice allows higher permissible stress than for other soft woods. However, unless one pays very high prices for clear Douglas Fir, other imported grades will be found very inferior and the permissible stress consequently limited.

Whitewood

Unsorted commercial whitewood is readily available and gives reasonably economical yields when stress graded. Therefore, although the beam must be of a slightly increased size, it does exactly the same work as Douglas Fir; in most instances at greatly reduced cost.

Stress grading, etc.

While machine grading produces a more economical yield than visual grading, machines have only just become available in this country for commercial use. However, this situation will probably soon change.

Apart from face-stamped Canadian Lumber Standards hemlock of doubtful quality, stress graded timber is not yet available from importers and merchants. For this reason timber has to be graded by the user and it is therefore not recommended that the higher quality grades are specified by the designer. 50 grade is accepted as the most economical proposition because readily available commercial grades fit into this category without regrading.

Laminating

Solid timbers of a cross section greater than 4 in. x 4 in. and 8 in. x 3 in. are prone to excessive splitting and are slow and expensive to kiln-dry. The use of a laminating process in which thin boards are glued together presents a solution to these and other problems when timber of large cross-section is required.

Timber is available commercially in lengths from 8 ft. to 24 ft., the average length in a parcel being approximately 16 ft. To produce longer lengths for laminating purposes, boards are jointed end to end by finger joints, namely multi-scarf joints.

An alternative method is to use ordinary scarf joints, although these are more wasteful of material. Kilning, planing, finger jointing and gluing and curing of laminated timber is carried out in a workshop where the humidity and temperature is carefully controlled and the machines are accurately set. Quality control checks are carried out at every step in the process and the final product is subject to shear block and bending tests on batch samples.

The resulting laminated member is stable and relatively free from shrinkage and splitting. Although laminated members are made at room temperature (no steam bending is used) curved beams can be formed at very little extra cost over a straight beam.

Glues

The glues most commonly used for laminating are:

Urea Formaldehyde	Interior Work
Resorcinol Resin	Exterior Work

The latter is fully weather proof and immune from chemical attack but is more expensive than Urea.

Laminated beam sizes

In common with steelwork, laminated timber beams and posts are available in certain economical sizes. No handbook exists, however, giving recommended sections; it is left to individual fabricators to produce the range of sizes they feel are necessary or economically viable.

For example, the maximum thickness of laminate recommended is 2 in. nominal, which, when planed to an acceptable surface for laminating purposes, will be 1¹³⁄₁₆ in. thick. Some fabricators, however, feel that this is too thick to produce satisfactory results and that in this thickness 'cupping' of the laminates might not be eradicated in the press. At Rainham Timber Engineering, for example, a 1¼ in. nominal laminate thickness is used, producing beams of 1¹⁄₁₆ in. increments in depth.

When a curved member is required to a small radius it is necessary to reduce the laminate thickness to avoid over-stressing when cold bending. Laminates of ¾ in. and 1 in. nominal thickness are used, producing finished laminate of ⁹⁄₁₆ in. and ¹³⁄₁₆ in. which are suitable for curving to radii of 6 ft. 9 in. and 8 ft. 6 in. respectively.

Finish

After laminating, glue extrusions are cleaned off and the members are planed to a thickness depending on the appearance grade specified. The British Standard on laminating recommends one of three grades.

1. Economy   2. Industrial   3. Architectural

1 is left as it comes from the press.

2 is reduced in width by ¼ in.

3 is reduced in width by ½ in. and sanded.

It should be noted, however, that while European, Scandinavian and Russian whitewood and redwood are probably full nominal widths, Douglas fir is notoriously scant in size

and, after kilning, an 8 in. wide board may be 7½ in. wide even before planing.

The following table gives the likely finish beam width to be expected from nominal board widths.

Timber	Board size	Beams	
		Architectural finish	Industrial finish
European Whitewood	4	3½	3¾
	5	4½	4¾
	6	5½	5¾
	7	6½	6¾
	8	7½	7¾
	9	8½	8¾
Douglas Fir	4	3	3¼
	6	5	5¼
	8	7	7¼
	10	9	9¼

It can be seen that Douglas Fir is particularly limiting in economic beam widths.

Fire resistance

Contrary to common belief, heavy timber structures offer a much better fire resistance than unprotected steel or concrete. Timber maintains its strength even at very high temperatures and when it burns it chars, forming a protective layer of carbon around the perimeter of the beam profile.

Thermal expansion and conductivity are very low and little deformation of the structure occurs. This eliminates the early collapse and subsequent pushing out or pulling in of walls which is often a feature of fires in steel-framed buildings.

Timber chars at the rate of approximately 1 in. in forty minutes and consequently members can be designed which afford a ½ hour-1 hour or other selected period of fire resistance.

A Class I rating of surface flame spread can be achieved by use of retardent paints and varnishes, such as *Albi*, *Timonox* and *Polycot*. For best results site application is recommended.

Moisture content

The moisture content of timber is expressed as a percentage of the water present to wood substance. In the growing tree it can be as high as 120% and seasoning or drying is achieved either by air-drying (natural seasoning) or kilning. Despite many old wives tales kilning may produce a more satisfactory result

Timber shrinks considerably on drying out and a drop of 30% in moisture content may reduce a 6 in. wide board to 5¾ in. Timber such as Douglas Fir or Hemlock from British Columbia may be shipped green and this can give rise to difficulties and expense for the purchaser.

Timber below 20% M.C. is reasonably immune from fungal attack.

As the moisture content decreases so timber increases in strength. For this reason C.P.112 gives two sets of permissible stress, one for timber 18% M.C. and below, and one for above 18%.

Laminated timber members will normally be about 12% M.C. when delivered to site but for certain jobs, i.e. hospital operating theatre structures the M.C. may be as low as 8%. Obviously steps must be taken to protect such members from wetting during erection and site storage. Members can be wrapped in polythene sheeting or coated with applications of lacquer to achieve this.

Preservation

The resistance of timber to acid attack, corrosive atmospheres, and humidity is extremely high and, therefore, maintenance costs are low. Many softwoods, however, are not naturally durable enough to give a long life when used externally.

For these cases preservation is necessary, and the methods available are by impregnation, immersion, and painting by organic solvents. The latter is most convenient and some commercial types are quoted.

Protim is beetle, fungus and termite proof, and fairly water repellent.

Sikkens Rhubbol is a protection which is particularly noteworthy as the coating stays pliable throughout its life and can, therefore, breathe with the timber.

Summary

Timber is extremely economical for single-storey construction, particularly when maintenance is considered. Particular advantage of this has been taken in industrialized building. Hyperbolic paraboloid roofs in timber have often made the concrete alternatives look unattractive from a price, speed of erection and appearance point of view. This is particularly true of non-repetitive work.

In GluLam work, however, too much emphasis has been made on an immaculate finish where its alternatives, i.e. steel or concrete, would not be scrutinized closely.

This has led to an inflated standard of production way above that which could be acceptable. (People are asking for joinery, not structural timber work.)

It would seem obvious that a greater involvement of consultants in timber engineering could create a larger market and hence reduce costs. The consultant, who ignores the possibilities of timber construction for both industrial and prestige buildings, is perhaps doing his client a disservice.

It is also obvious that anyone who insists on too high a grade of timber for unrealistic reasons will continue to pay far more than he needs.

Sway analysis with particular reference to computer modelling techniques

Bob Emmerson

Sway is the horizontal deflection of a structure under imposed lateral loadings.

A structure, sized only by allowable stress design, may be unacceptably flexible. Excessive sway can damage glazing, partitions, etc., and cause distress to the structure's inhabitants. (It should be noted here that the comfort of the inhabitants is really dependent on the magnitude and duration of the structure's acceleration.) Our analysis will be a static one using static loadings; however structures sized by these methods, and with the undermentioned sway limitations, have been constructed and proved satisfactory in service, for example, the Empire State Building in New York.

American consultants generally design for a maximum allowable sway equal to $\frac{1}{500}$ of a structure's height.

If we use $\frac{1}{500}$ of a structure's height as our design sway criterion for office and residential structures—then a structure with a 10 ft. (3 m) storey height will have a maximum inter-storey sway of the order of ¼ in. (6 mm). If we consider the low probability of recurrence of maximum design wind velocity then this should prove acceptable for partition and glazing design and should also ensure an acceptable comfort factor for the structure's inhabitants.

The sway of a structure may be analyzed by hand. This process is time consuming and imprecise.

The *complete* structure, with initial design sizes, may be inserted into a plane frame program or a space frame program and run on a computer. The member sizes can be progressively adjusted over a series of computer runs until both stress and sway limitations are satisfied. This process will give accurate final results; nevertheless it is excessively time consuming and uneconomical.

A simple design technique exists for modelling a complex structure into one with a smaller number of joints and members. For example, a framed 30-storey structure with 10 bays can be modelled into one with three storeys and 10 bays. The member properties of the actual structure are lumped together to give the member properties of the model, and the wind loadings on the actual structure are lumped together to give the wind loadings on the model.

The modelled structure method has in the past given a sway prediction of the actual structure to an accuracy of 5% to 10%. The modelled structure method can also be used to predict the load sharing of individual lateral load resisting 'elements' within the structure, i.e. separate model frames can be linked together in series. (Fig. 2).

The modelling principle is illustrated in Fig. 1. If m is the model ratio equal to the number of floors modelled into one, then the lumped member properties are as follows:

$$Agm = \sum_{1=1}^{m} Agi$$: the cross-sectional area of the girder for the model equals the sum of the actual cross sectional areas modelled.

$$Igm = \sum_{i=1}^{m} Igi$$: the moment of inertia of the girder for the model equals the sum of the actual moment of inertias modelled.

$Acm = Aci$: the cross sectional area of the column for the model equals the cross sectional area of the actual column.

$$Icm = m\sum_{i=1}^{m} Ici$$: the moment of inertia of the column for the model equals the sum of the actual moment of inertias modelled multiplied by the model ratio.

$$Adm = \frac{(Sdm)^3}{(Sd)^3} \frac{Ad}{m}$$: the area of diagonal for the model equals the ratio of the model length of the diagonal to the actual length cubed and multiplied by the actual area divided by the model ratio. The diagonal is assumed to be pinned at both ends. The formula holds for 'K', 'X' and diagonal bracing.

$$Pm = \sum_{i=1}^{m} Pi$$

The output from the computer run gives forces and moments in the modelled structure. These forces and moments in the modelled members have no direct relationship to forces and moments in the members of the actual structure with the following exceptions:

(1) Column axial loads
(2) Support reactions

If the forces and moments in members of the actual structure are required, then a portion of the structure may be left unmodelled without altering the accuracy of the sway prediction. For example, it is possible to leave the lower two or three storeys unmodelled.

Modelling enables the design process to be simplified and greatly speeded up as follows:

The structure is first approximately sized by hand[1] and then modelled as described above. The relevant data is then inserted into a suitable program which is run on the computer. If sway performance is unsatisfactory after the first run, then the lumped member properties are adjusted by inspection, without back reference to individual member properties. The program is then re-run with the new lumped properties. If this run is satisfactory, then the lumped member properties can be re-allocated to individual members by inspection. A detail run of the complete structure can now be made to check the model's drift

prediction and to enable detailed design to be made of individual members.

A recent illustration of this modelling principle is OAP Job No. 2985/T. The 22-storey office block (Figs. 3 & 4) at Kingston, Jamaica, is only part of a comprehensive redevelopment of the waterfront. The development comprises shops, garages, apartments and an hotel.

It was clear from the outset that lateral loading would dominate the design of the structure. Allowance had to be made for hurricane wind loading up to 100 lb/sq. ft. (4.79 kN/m²) and also for an earthquake of intensity 8½ on the Modified Mercalli Scale. Fortunately it was

not required to consider these to act simultaneously!

A conventional framed structure required very large column and beam sizes. These restricted architectural planning, reduced rental area, and increased the total building height. A building, relying solely on core stiffness to resist lateral loading, required an excessively large core area. This again reduced rental area and planning flexibility.

A satisfactory solution was to use the facade as the lateral load resisting element. The structural sizes would then have a minimal effect on internal planning flexibility. Initially
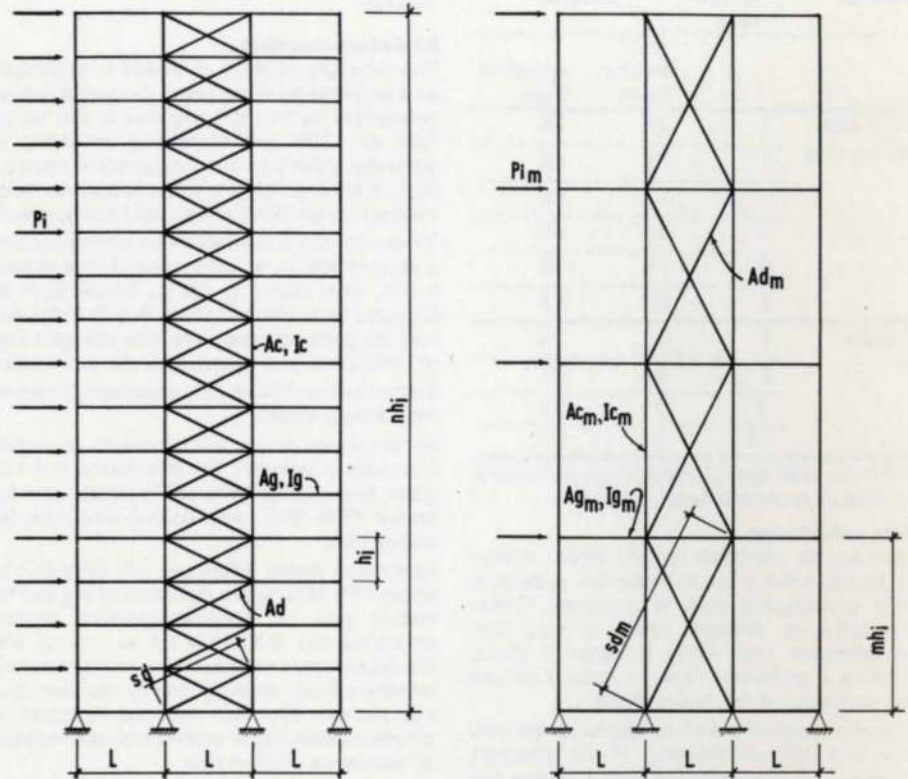
a) Theoretical structure
b) Model structure

Fig. 1
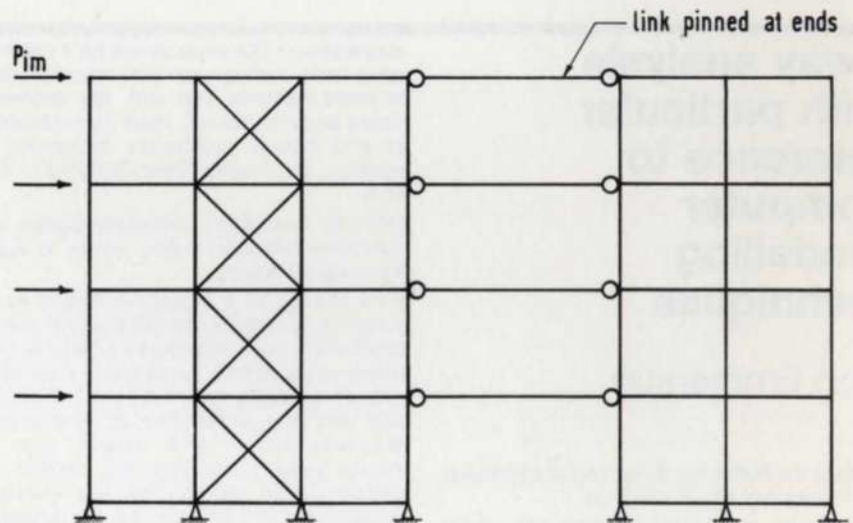Modelling principle
(Drawn by: Mara Medenis)

Fig. 2
Model structures linked in series
(Drawn by: Mara Medenis)

a cross-braced truss on all four faces was considered. This truss would behave in a similar manner to a thin walled 'tube' under the application of lateral loading. An example of this type of structure is the John Hancock building in Chicago. The trussed 'tube' solution was rejected on two counts:

(i) Aesthetic

(ii) Complicated detailing

It was thought that the 'thin walled tube' behaviour could still be obtained if a frame of sufficient rigidity was used. Column centres of 10 ft. 8 in. (3.25 m) were agreed. An additional spandrel was inserted at approximately mid-height between floor spandrels. (Fig. 6). The additional spandrel stiffened the frame and reduced both moments and shears in the column and spandrel. This spandrel produced the additional bonus of acting as a sun-break or 'brise soleil', thus reducing heat gain within the building.

A preliminary and very approximate estimate was made of member sizes by hand. Three conditions were considered:

1. Dead Load + Live Load

2. (Dead Load + Live Load + Wind) x 0.8

N.B. This is equivalent to a 25% increase in permissible stresses.[2]

3. (Dead Load + Live Load + Earthquake) x 0.75

N.B. This is equivalent to a 33% increase in permissible stresses.[2]

This preliminary design assumed:

(a) A two dimensional frame action to deduce the shear forces and moments within members.

(b) A three dimensional 'thin walled tube' behaviour to deduce the column axial loads.

A mathematical model was made of the structure using these preliminary sizes. The double-storey height entrance hall was left unmodelled and the 20 storeys above were modelled into four storeys using a model ratio, m, of 5.

This three dimensional model was inserted into a space frame program and lateral loading was applied. The computer output showed significant 'tube' action and predicted an acceptable drift. Fig. 8 shows a plot of column loads at ground floor level due to wind overturning.

A detailed check was now required of member forces and moments. The complete three dimensional structure, even with economies resulting from symmetry, greatly exceeded the capacity of our computer.

The solution was to insert one side frame into a plane frame program. The stiffening of the tube flanges was simulated by adjusting the cross-sectional area of the corner column. The cross-sectional area was adjusted so that the vertical deformation of the corner column under direct load equalled that of the 'tube' flanges under direct load, i.e. the total forces in the flanges (see Fig. 8) were applied to a corner column. This column was then given sufficient cross-sectional area so that its deformation under these applied forces equalled that of the 'tube' flanges given in the output from the 'tube' model structure's computer run.

The results of this computer run verified the sway prediction and gave member forces and moments for the final design.

It is not known whether the building will be constructed. Nevertheless all members of our team agree that this has been one of our most rewarding analytical experiences.
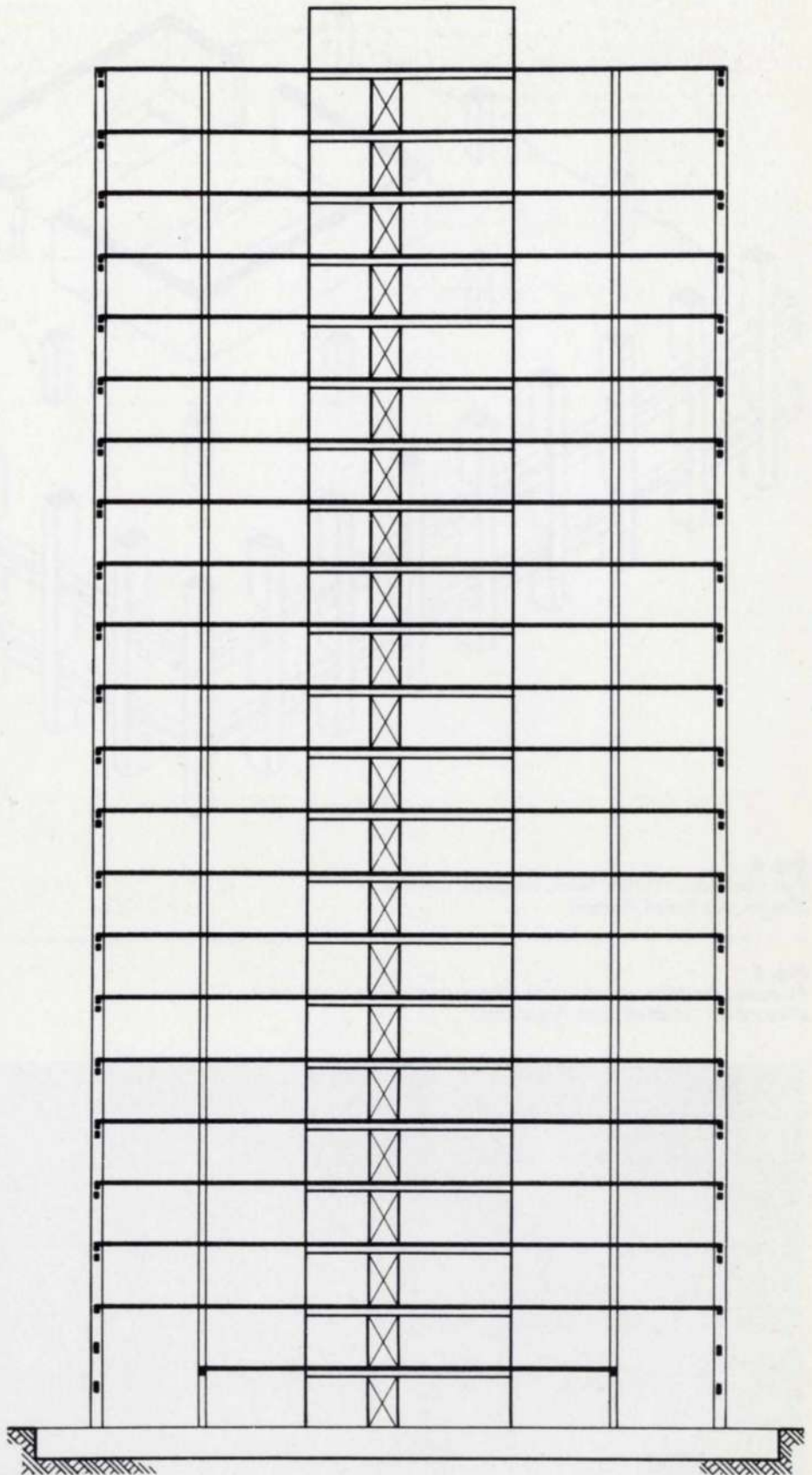
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[2] Design for load conditions 1, 2, and 3. was on the basis of not exceeding permissible design stresses. It is, therefore, more convenient to reduce the loadings for load conditions 2, and 3, than to increase the permissible stresses.
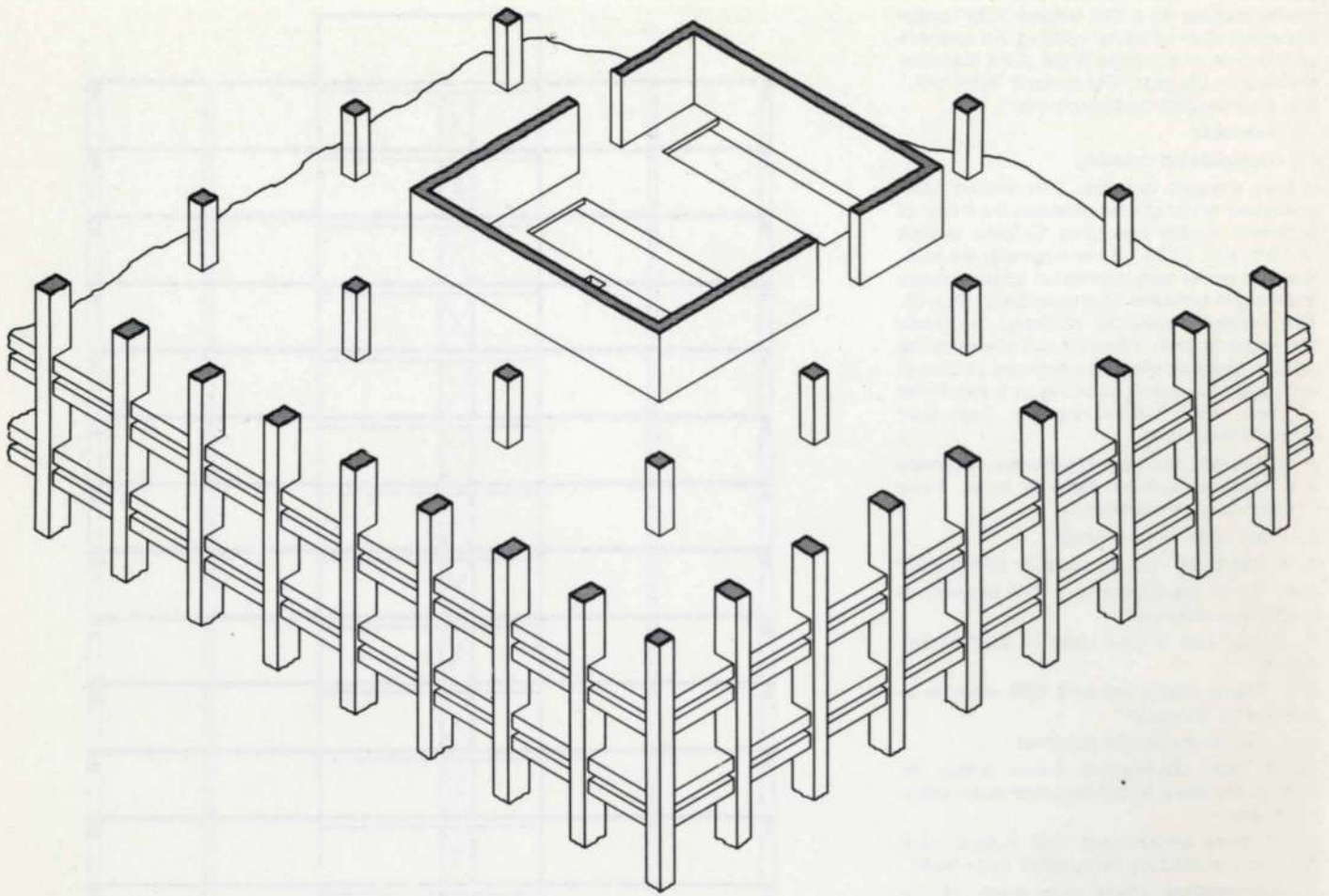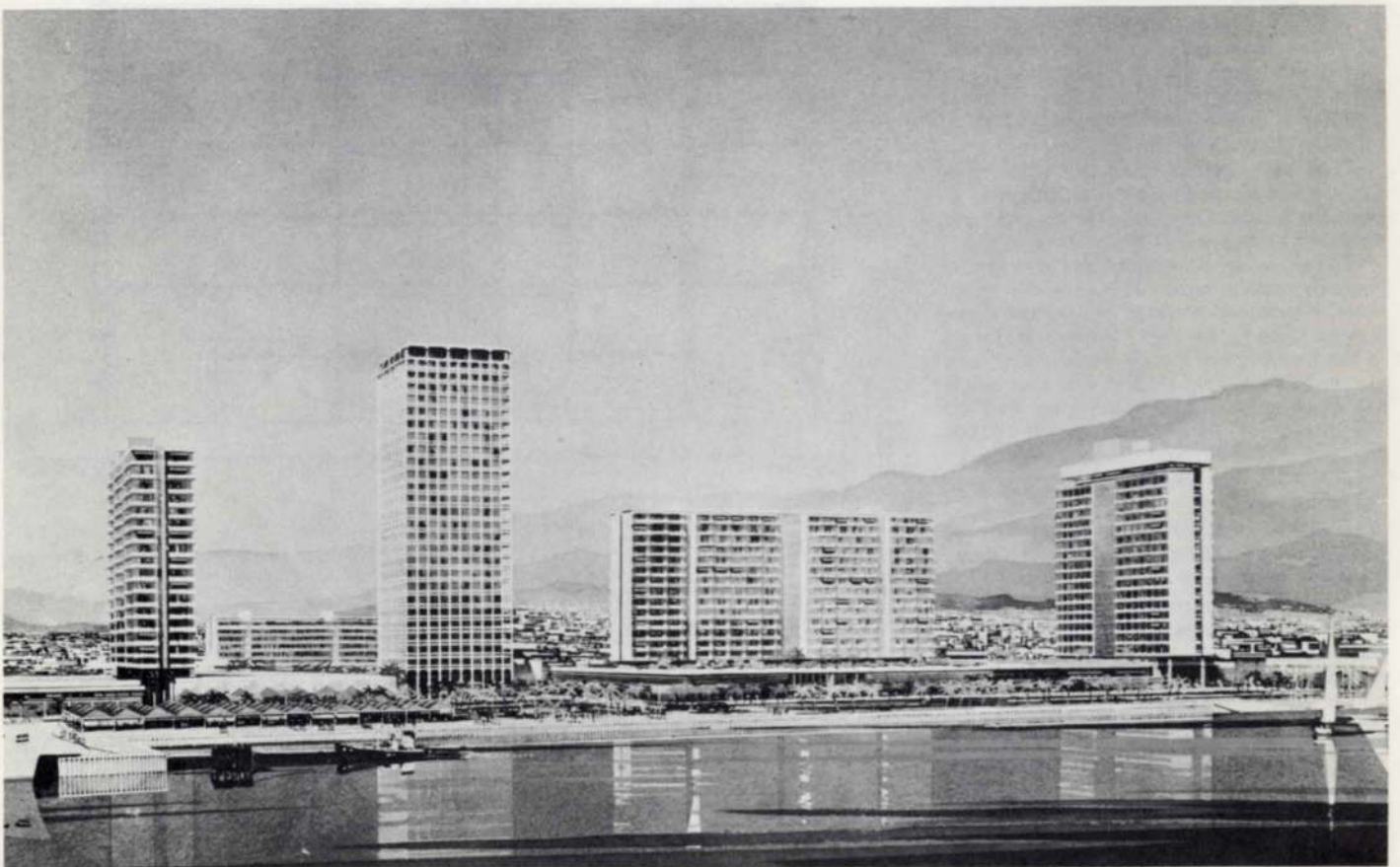

Fig. 3
Section through office block, Kingston, Jamaica
(Drawn by: Mara Medenis)

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Fig. 4
Part axiometric of office block, Kingston, Jamaica
(Drawn by : Susan Pickard)

Fig. 5
Proposed Kingston waterfront development showing tower block
(Illustration : Michael Lyell Associates)

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APPENDIX A

Approximate analysis of structures for lateral loading.

The most convenient design method is suggested by the Portland Cement Association of America and is known as the joint coefficient method.

If member stiffness

$$K = \frac{\text{Moment of inertia}}{\text{Length}}$$

Then the joint coefficient v

$$= K \text{ for column } \frac{(\text{sum of K values for adjacent girders})}{(\text{sum of K values for all adjacent members})}$$

If we have a total lateral wind force W acting on a frame above level n then the proportion of that force resisted by a column at joint A within the frame is equal to:

$$\frac{Va}{\sum Vn}$$

Points of contraflexure are assumed to act at mid-height of column. The design process is clearly illustrated by Fig. 7.
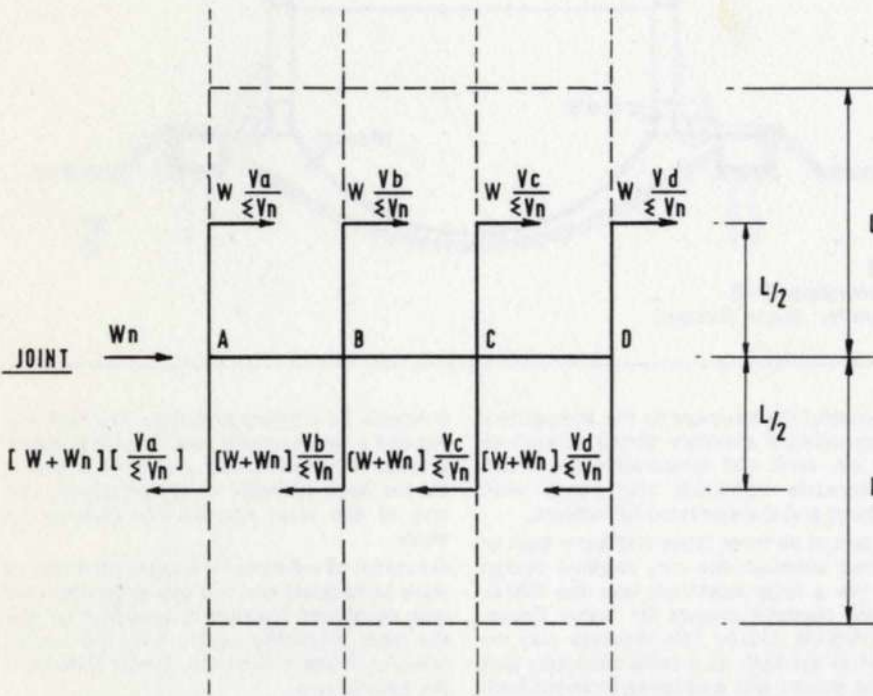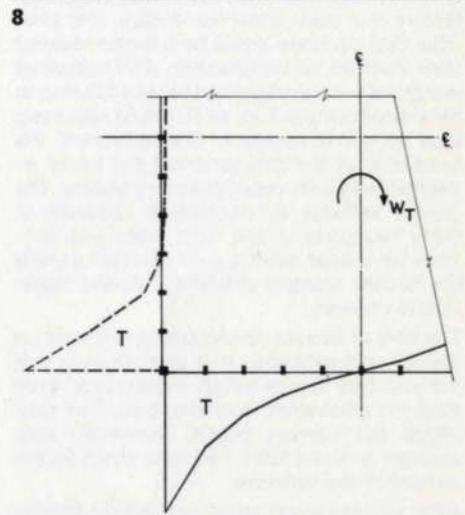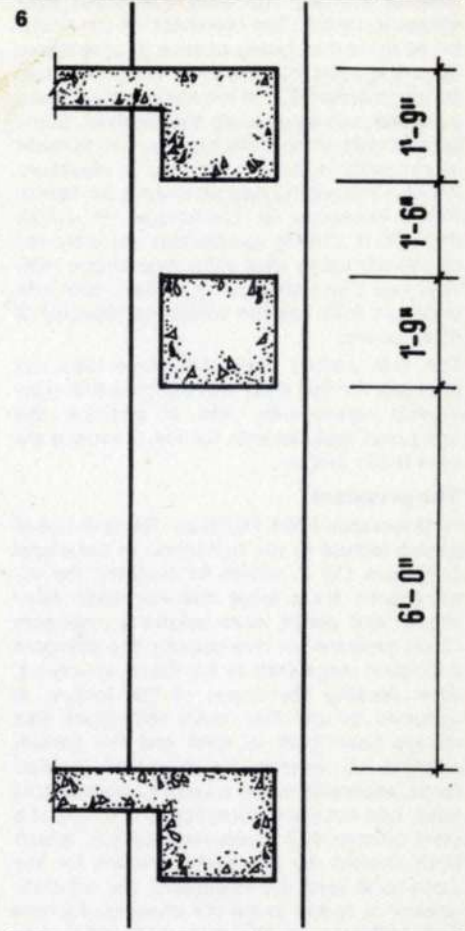
Fig. 6
External column elevation,
office block, Kingston, Jamaica
(Drawn by: Mara Medenis)

Fig. 7
Joint coefficient method.
Assumed distribution
of wind shears
within frame.
(Drawn by: Mara Medenis)

Fig. 8
Plot of external
column loads at
ground floor level
due to wind
overturning moment
(Drawn by: Mara Medenis)

7

8

A proposal for a 400 ft. radio telescope in reinforced concrete

Alistair Day

Introduction

Our knowledge of the universe comes from our ability to see through the atmosphere to the space beyond. This transparency of the atmosphere to electro-magnetic waves occurs in two parts of their spectrum; the range of visible light plus a little bit on either side, and the range of some radio wavelengths. The remaining wavelengths would be absorbed by the atmosphere, which is just as well as some of them, notably gamma rays, would be distinctly harmful to us. Thus, these two 'windows' in the atmosphere allow the Earth's surface to be touched by these two types of radiation only.

We can only 'see' things which emit the wavelengths we call 'visible' because nature evolved eyes which are sensitive to these wavelengths only. There is no reason, in principle, why eyes which are sensitive to radio wavelengths should not have evolved, but the practical reason seems to be the very large size which would be required. These eyes would be disproportionate to the size of the creatures which have developed on Earth, and also they would not have a good enough resolution for our purposes.

The visible part of the spectrum has been used from the earliest days to study the universe. A lot of information about the sky can be obtained with the unaided eye, but in 1608 a Dutchman named Hans Lippershey happened to line up two spectacle lenses at a church steeple and the subsequent development of the refracting telescope, and its later cousin, the reflector, has provided basic data for successively more advanced theories on the evolution and form of the universe.

Radio waves from space were discovered in 1931 by an American communications engineer named Karl Jansky. In a very elegant observation, he noticed that the background 'noise' in an experiment he was carrying out had a periodic variation which coincided with the sidereal day and not the solar day. This meant that the radio waves forming the noise were coming from space and not the sun. After World War II the technology of radio telescopes advanced rapidly and several different types were developed.

The most well known type is the parabolic dish, which can be either fully steerable, as at Jodrell Bank, partially steerable, like the 300 ft. (91 m) installation at Green Bank, West Virginia, U.S.A., or fixed, as at Arecibo, Puerto Rico. Here, the Americans lined a 1000 ft. (305 m) natural hollow with a reflecting surface and hung a receiver at the focal point above it, anchored on the surrounding hills. (In contrast, one of the first small radio telescopes was made of chicken wire on scaffold poles!)

The other main type consists of two long arrays of dipoles built at right angles in the **15**

form of a cross; the arms sometimes kilometres in length. The drawback of the Arecibo bowl, is that, being passive, it only traces out a line across the sky once every 24 hours, although about 20° of movement is obtained on either side by moving the receiver. Some later models of the cross type have arms made of parabolic cylinders, movable in elevation. Another interesting development is Sir Martin Ryle's telescope at Cambridge, in which three 60 ft. (18 m) paraboloids are mounted on railway tracks over a distance of one mile. This has the resolution of a dish one mile wide but only has the collecting capacity of 60 ft. bowls.

The first Jodrell Bank telescope was not intended for the short wavelengths that now interest astronomers, and, in addition, the functional requirements for the telescope are now better known.

The proposal

In November 1964 Professor Bernard Lovell gave a lecture to the Institution of Structural Engineers (1) in which he outlined the requirements for a large diameter radio telescope, and asked what solutions engineers could propose for overcoming the stringent deflection requirements for these structures. After reading the report of the lecture, it occurred to me that radio telescopes had always been built in steel and the greater stiffness of concrete for short-term applied loads, especially when related to normal cost rates, had not been examined. If the cost of a steel column and a concrete column, which both shorten by the same amount for the same axial load, are compared, the concrete column is found to be the cheaper. As cost and stiffness are the two most important factors in a radio telescope design, it is possible that concrete could be a better material than steel for its construction. The increased weight of a concrete structure would seem to be a disadvantage but, as the most important load on the telescope is the wind load, the behaviour of the structural material under an applied load is its most important quality. The greater stiffness of concrete is apparent in other structures where both steel and confrete have been used, e.g. in pressure vessels cor nuclear reactors and in microwave transmission towers.

The idea of floating the structure followed on almost automatically, as it was obvious that the structure would weigh a great deal more than the equivalent steel structure. This simplified the support bogies immensely and, contrary to initial fears, had little effect on the control of the structure.

After we had made some preliminary studies it appeared that the use of concrete as the main structural component was a viable alternative to steel, so we approached Jodrell Bank to find out if there were more detailed design requirements available than the outline given in the paper.

We were given the specification which was proposed for the next telescope for Jodrell Bank and, when we had examined the required performance, we decided to make a feasibility study for a 400 ft. (122 m) telescope, as our earlier work indicated that the specification could be met.

Structural types

As large movable radio telescopes with diameters greater than 250 ft. (76 m) had not been built, the way the design would evolve for a 400 ft. or greater diameter telescope was a matter of conjecture. At the time there were three main approaches to the problem of providing a reflecting dish which did not deform under wind load, etc.

1. Protecting a light structure in a radome.
2. Exposing the structure to the atmosphere and providing a rigid structure to prevent excessive deformations due to ice, wind and temperature loads.
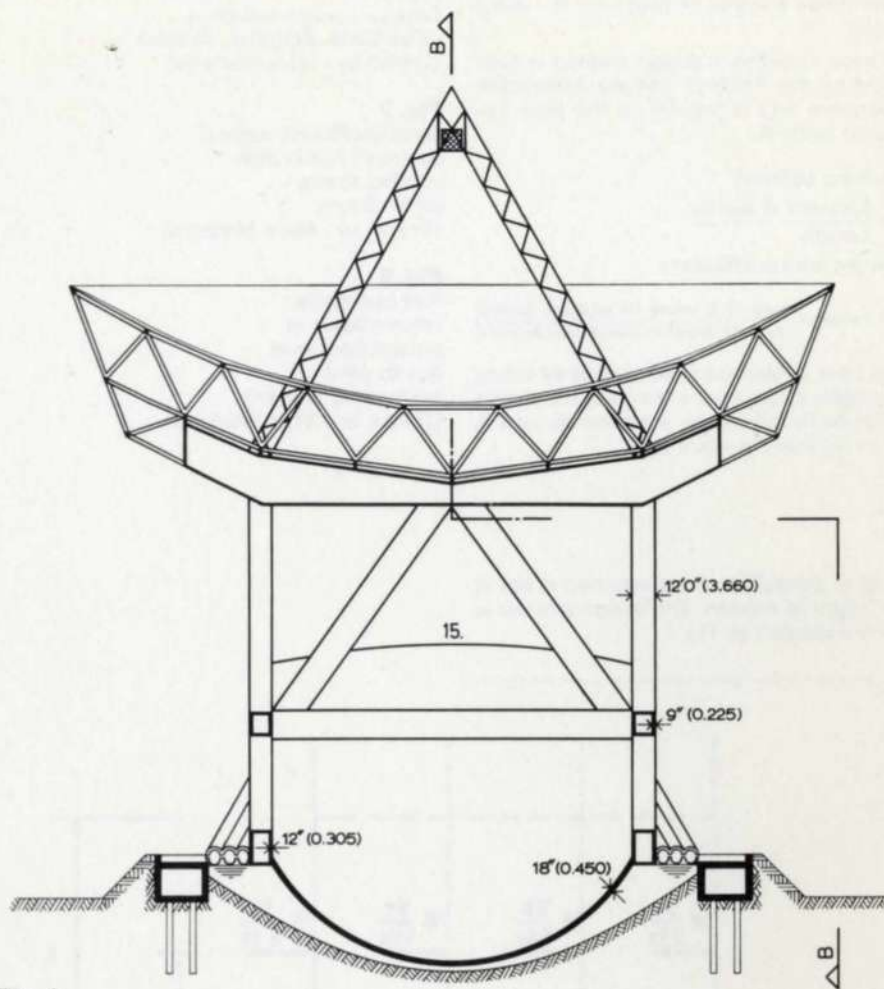

Fig. 1
Side elevation B—B
(Drawn by: Susan Pickard)

3. Exposing the structure to the atmosphere and providing a structure strong enough to resist ice, wind and temperature loads, but with separate adjustable skin panels with monitored and compensated deflections.

Examples of all three types had been built or designed although the only detailed design made for a large telescope was the 600 ft. (183 m) diameter project for Sugar Grove, West Virginia, U.S.A. This structure was intended to act both as a radio telescope and satellite tracker and was being financed from the American defence budget. It was originally intended as a structure of the second type but the weight proved too great for the foundations, which had been built to carry a load derived from a preliminary design. To reduce its weight, it was redesigned as a structure of the third type but by then its defence role had become obsolete so it had to be abandoned due to lack of financial backing.

The diameter of the next telescope which was intended for Jodrell Bank was 400 ft. This was nearly twice the diameter of the existing largest fully-steerable telescope with a dish area proportionately greater, and, in addition, a greatly improved performance was required.

This could well have meant that the solutions evolved for the current models were not necessarily the correct ones for a large telescope, analogous to the fact that different types of bridges are required for varying span-widths. Thus the design study made by Paul Weidlinger for a structure in a radome depended on a concept radically different from the existing designs. As well as that study, a number of other feasibility studies were being made in America to test new concepts for a future telescope. The fact that we had a new concept which could possibly provide an answer to the problems produced by the large diameter of the telescope was one of the main reasons for making our study.

A number of schemes for a concrete structure were considered and the one described here was developed because it appeared to give the most promising results with the limited amount of effort available. It was initially of the second type.

Design specification

The main requirements of the design specification were:

Elevation movement 5° to 95°

Azimuth movements 360° +60° overlap

Wind speed for normal operation up to 25 m.p.h. (40 km/h)

Maximum speed for moving reflector up to 45 m.p.h. (72 km/h)

Survival wind speed up to 100 m.p.h. (160 km/h)

A ratio of 0.4 between the focal length and the diameter was assumed, as this was the ratio preferred by Jodrell Bank.

The deflection of the dish was specified in a way which will be described later.

Development of the structure

Because of the great weight of the concrete structure, the method which was used to carry this weight had a fundamental effect on the geometry of the structure. With a steel structure it is possible to carry the whole weight on groups of bogies but, if they are used for the concrete structure, their number becomes excessive and the whole thing is

too cumbersome and expensive. The simplest way of carrying a large weight is to float it. This was, therefore, the method which we used in our design. Practically the whole weight could be carried by buoyancy so that the bogies were mainly used to locate the dish and only had to carry sufficient weight to overcome the wind load at wind speeds occurring during operation. It was found that only 5% of the weight had to be carried on the bogies.

A number of different forms of the float were looked at. The float geometry was governed to a great extent by the movement required both horizontally and in elevation. Some forms of float which gave the required buoyancy were either not strong enough to carry the load, or could not be proportioned so that bogies could run on rails attached to the float to give the required movement. A float which was shaped as part of a toroid appeared to be the most suitable form as it met all the requirements of strength, buoyancy and geometry.

The position of the dish, relative to the toroidal float, depends on the elevational movement required. Once the size of the toroid and the position of the dish relative to it had been fixed, a structure connecting the two of them was needed. Again a number of different types of structure were considered, but the simplest form, which consisted of two plane frames lying in vertical planes so that they did not move out of this plane as the telescope rotated in elevation, appeared to be the best solution. These frames were made in a triangulated form so that the loads were carried as axial forces. When combined with the floats they formed an extremely stiff structure.

The reflecting surface had to be metallic and, in the design we evolved, the girders immediately behind the reflecting surface were in steel, although concrete was considered for them at one stage.

Proposed structure

The design as it eventually emerged is shown in Figs. 1 to 4 and consisted of nine main components:

1. *Foundation ring beam*. The foundation was a box ring beam of 235 ft. (72 m) inside diameter containing the pond in which the structure floats. A vertical support for the bogies was obtained by a pair of concentric rails on the top of the foundation.

2. *Bogies*. Four support structures containing upper and lower sets of bogies were supported on the foundation ring beam and supported the moving structure on the upper bogies. The bogies had wheels in both vertical and horizontal planes giving a vertical support and horizontal restraint to the moving structure.

In plan the bogie housings were at the four corners of a rectangle with the wheels of the lower bogies lying in concentric circles and those of the upper bogies lying in two vertical planes.

3. *Float*. The toroidal shell was 18 in. (450 mm) thick and stiffened by curved box beams lying in vertical planes along two sides of the shell. Rails on the underside and outer face of the stiffening beams located the float on the bogies.

The radius of the stiffening beams was governed by the geometric relationship between the radius of the foundation beam and the width of the toroid.

The major and minor radii of the toroid were determined by the buoyancy required and the radius of the edge beams. The arc of the major radius was determined by the elevational rotation required and the angle subtended by the water line.

4. *Main concrete frames*. Two concrete frames lying in the vertical planes containing

2

Key
1. Laboratory
2. Steel leg
3. Reflecting skin
4. Main girders
5. Main beams
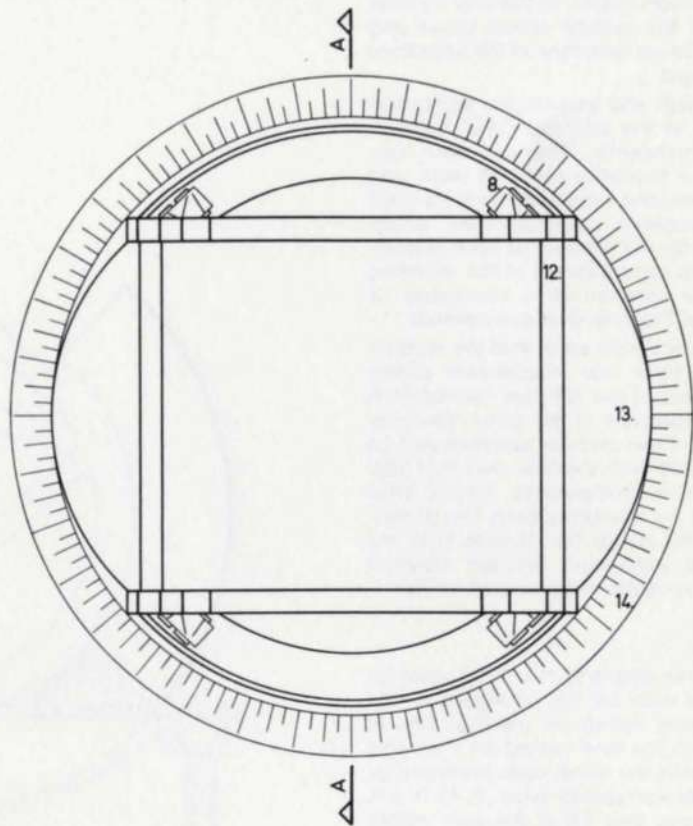6. Float stiffener
7. Float
8 & 9. Bogies
10. Ring beam
11. Piles

0   100   200   300   400 ft.
0   25   50   75   100   125 m.
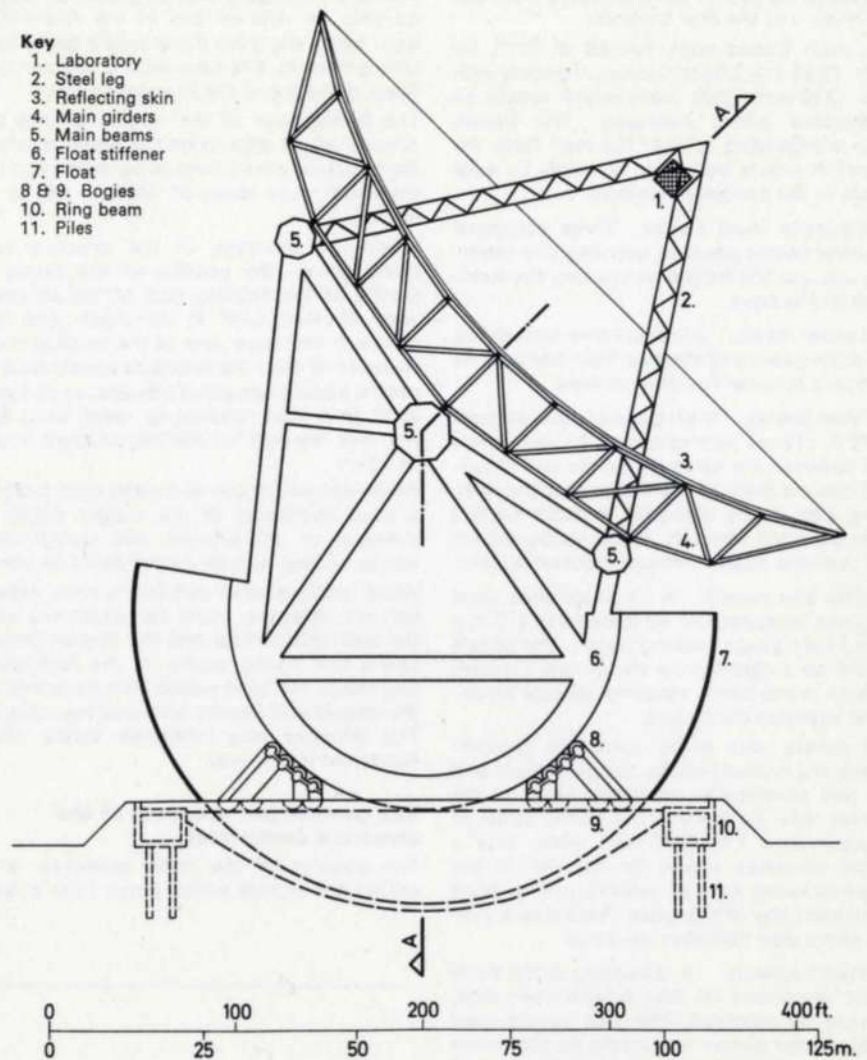
3

Fig. 2
Section A–A

Fig. 3
Plan of float and foundation

(Drawn by: Susan Pickard)

the stiffening beams were provided between the floats and the dish supports.

The main frames were formed of 12 ft. by 12 ft. (3.65 m x 3.65 m) hollow members with 9 in. (230 mm) thick walls which would be prestressed where necessary. The frames were triangulated so that the load from the reflectors would be carried primarily by axial forces in the concrete members.

5. *Concrete main beams.* Three octagonal concrete beams spanned between and extended beyond, the frames and carried the steelwork of the bowl.

6. *Lateral frame.* A lateral frame connecting the main beam and the two main frames was provided to resist lateral wind load.

7. *Main girders.* Main girders in tubular steel at 40 ft. (12 m) centres across the dish spanned between the concrete beams and extended beyond them to the diameter of the dish. They were to be triangular in cross section with the apex upwards, being supported on the concrete beams through adjustable jacks.

8. *Skin and panels.* A 14 gauge sheet steel skin was supported on 40 ft. by 40 ft. (12 m x 12 m) light gauge backing panels. The panels would be supported on the girders through flexible connections allowing relative movement between the girders.

The panels were to be curved to approximately the correct radii for the paraboloid and the skin attached by adjustable studs to the correct radii. By forming the skin in strips of approximately 1 ft. (300 mm) width only a single curvature would be required in the strips allowing ease of manufacture without significant loss of tolerance. A solid skin over the entire dish had been assumed.

9. *Focal support.* A laboratory at the focal point, supported on four tubular steel legs, was to be provided. The legs would pass between the girders and would be supported directly at the intersection of the frames and outer pair of main beams. In this way no local distortion of the surface would occur and there would be no restriction of the laboratory size and weight.

For both design and analysis the eight main components of the structure could be considered independently. This was very convenient for a feasibility study as each part could be designed separately and standard computer programs could be used, where appropriate, for the analysis of each section. Deflections of various points in the reflecting surface were obtained by a summation of deflections of the individual components.

At first sight it would seem that the division of the steel bowl into independent girders lost the benefit of the stiffness derived from the double curvature of the bowl. However the stiffness of the concrete structure was so great compared with the steel skin that little improvement in performance would have been gained by interconnecting the girders. In addition the facility for adjustment of the skin without interaction between different parts of the structure compensated for this.

Movement

Elevational movement of the dish would be produced by rails on the underside of the stiffening beam rolling on the four sets of upper bogies. The load carried on the bogie wheels to resist the wind loads produced by the maximum operational wind of 45 m.p.h. (72 km/h.) was only 5% of the dead weight of the structure. Because of this a friction drive using the wheels of the bogies could not be used so the force to move the structure would be provided by a rack and bogies pinion. The rack for the elevation movement would be in two arcs fixed to the underside of the edge beams of the toroid. Pinions on top of the bogies of each of the four bogie units would move the structure.

Azimuth movement was obtained by bogies running on rails on top of the foundation ring. Again the drive force would be by rack and pinion+ in this case with a circular rack fixed to the top of the foundation ring.

The horsepower of the motors driving the pinions would depend on the speed at which the structure would have to be driven and the maximum wind speed at which it had to be moved.

When the geometry of the structure was being drawn, the position of the centre of gravity of the floating part of the structure was adjusted until it coincided with the centre of the major axis of the toroidal shell. Because of this, the structure would float in neutral equilibrium about this axis, so no force, apart from that required to resist wind forces, was required to keep the structure in any position.

To eliminate backlash at the elevation pinions, a small imbalance of the weight could be arranged or, alternatively, two motors with one providing counter torque could be used.

When extreme wind conditions were expected, the structure could be positioned with the dish axis vertical and the bogies located above four strong points on the foundation ring beam. The load would then be jacked off the wheels and directly onto bearing surfaces. The structure was inherently stable when supported in this way.

The geometrical accuracy of the structure under load

The purpose of the radio telescope is to collect the signals which arrive from a radio source in space and reflect them to a point where they may be picked up by a receiver. The distance between the source and Earth is so great that the radio signals arriving at the telescope may be considered to lie in parallel rays, so that, if the surface of the dish is in the form of a paraboloid, all the reflected rays will meet exactly at the same point, the focal point. In an actual telescope it is impossible to produce an exactly true paraboloid; a certain tolerance must be allowed in the erection, and the wind and gravity loads acting upon the surface will distort it. Small variations on the surface will cause some of the reflected rays to miss the focal point and so be lost to the receiving apparatus. There is some tolerance available in the amount by which the rays miss the focal point, but obviously the greater the surface varies from the paraboloid, the greater the amount of the signal which will be lost. If the deflected form of the surface is plotted for any given loading condition then the distance between the ideal paraboloid and the actual surface can be derived for a number of points. If the values of the distances are squared, the mean value of the squares can be found, and the square root of the mean taken. Thus the root mean square (R.M.S.) of the distortion of the surface is found. It can be shown that the loss of signal is a function of the R.M.S. value so that in any specification for a radio telescope the maximum R.M.S. permitted is always what is stated. For the 400 ft. telescope at Jodrell Bank, the specified R.M.S. value was 0.31 in. (8 mm) in a 25 m.p.h. (40km/h.) wind.

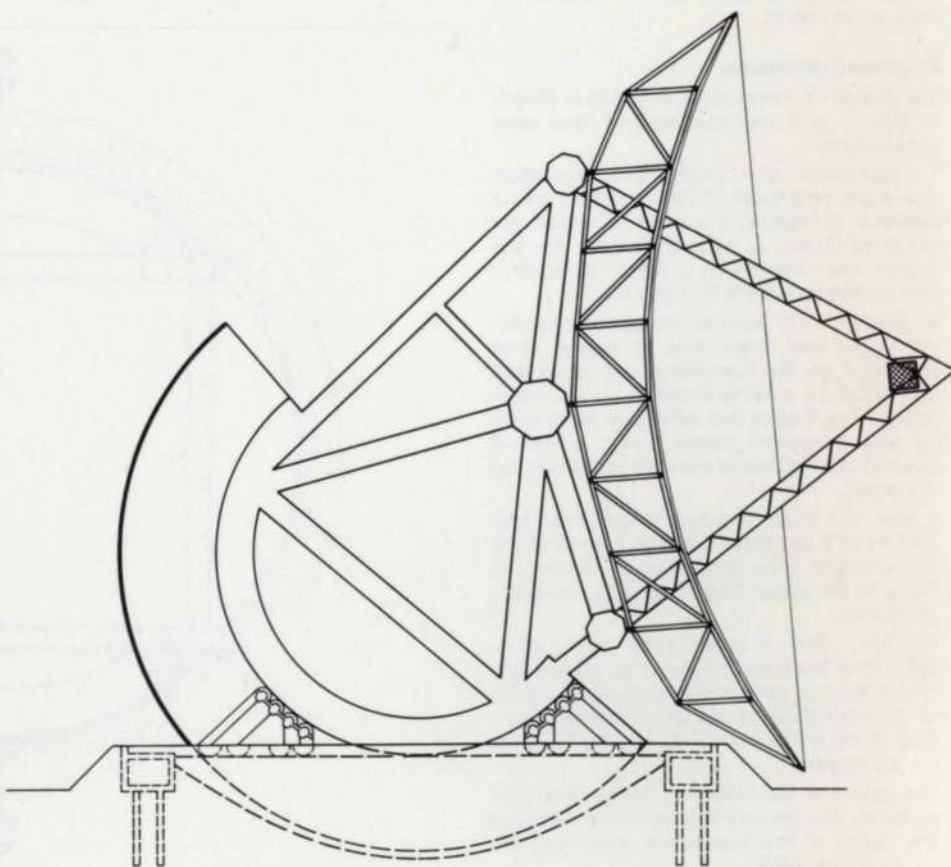As a telescope rotates in elevation, the direction in which its own weight acts on the

Fig. 4

Dish at lowest elevation
(Drawn by: Susan Pickard)

structure will change so that it will have a different deflected form at different elevations.

This is analogous to a beam which is rotated about one end. If the beam is assumed to be truly straight when it is standing on one end in a vertical position, then when it is rotated to the horizontal, its own weight will appear as an increasing lateral load, causing the beam to bend laterally, the more the nearer it gets to horizontal.

The telescope was assumed to be built, and its surface adjusted to as near a true paraboloid as possible, with its axis inclined at 45° to the horizontal. Then when the telescope was rotated until its axis was horizontal or vertical, the same amount of deflection due to its own weight would occur, for the two extreme positions, but with the deflections on opposite sides of the initial position.

In the design which we evolved, we found that the R.M.S. tolerance could not be maintained at the extreme elevations of the telescope without adjusting the relative positions of the steel girders and the skin supported on them. To obtain this adjustment, jacks were assumed at each of the three points where the steel girders were supported on the three octagonal concrete beams. To find the paraboloid from which the deflections would be measured, the parabola which most nearly fitted the skin supported by the central girder was used to generate it. The deflected form of the surface supported by each of the other girders was then placed to fit the paraboloid as closely as possible and the R.M.S. calculated from the deviations from the paraboloid.

It was found that the upper tip of the central girders deflected excessively under gravity loads, so a secondary set of jacks to adjust the position of the ends of the panels supported by the tip of these girders was used.

By using these jacks, the deflection of the concrete frame and the main beams (that is, all deflections of the structure up to the point of support of the jacks) could be eliminated by predicting the displacements and counteracting them by appropriate adjustment of the jacks.

When calculating the R.M.S. of the deflections the tolerance required for erecting and positioning the skin of the dish was arbitrarily assumed to be additional to any gravity movements. It was found that the R.M.S. of the maximum deflections for gravity loads was 0.10 in. (2.5 mm) and would occur at the maximum and minimum elevations. Between the extreme elevations the R.M.S. would be smaller.

Without a wind tunnel test the exact distribution of the wind pressure on the dish could not be known. However, for this feasibility study, two extreme pressure distributions were assumed covering any distribution which might occur, one being a uniform pressure over the whole dish area and the other a distribution in which the wind pressure acted upwards on one half of the dish and downwards on the other half. For both these loading conditions the wind was assumed to blow in either direction.

As the self-weight, or gravity load, was a constant load and its direction for any elevation known, it was possible to use the jacks to adjust the positions of the girders to obtain the best surface. However wind load is quite arbitrary in direction so no compensating adjustment of the surface can be made in advance. Thus the whole of the structure had to react passively to the wind load and hence all the wind induced deflections from the level of the foundations upwards are included in the deflection of the surface.

To find the deflected form of the dish at any position, that wind load distribution which would increase the gravity deflection was assumed to occur, and R.M.S. of the increased deflections found. The asymmetric

pressure distribution was found to be the most severe condition.

Because the steel in the structure would heat up or cool at a different rate from the concrete, and because the sun could be shining on the dish with the remaining part of the structure in shadow, a differential temperature between the steel and the concrete could be expected. This differential is assumed to be up to 10°C. and to allow for maximum variations, two conditions were used, one with the whole dish at 10° above or below the concrete temperature and one with only half the dish at the differential temperature.

In the same way as the wind can blow in any direction, the temperature differential is ambidirectional, and no predetermined adjustment of the surface can be made.

For the various loading conditions the worst R.M.S. deflections of the surface which we found were:

Loading Conditions	R.M.S.
1 Gravity	0.10 in. (2.5 mm)
2 Gravity +25 m.p.h. wind	0.15 in. (3.8 mm)
3 Gravity +45 m.p.h. wind	0.33 in. (8.4 mm)
4 Gravity +25 m.p.h. wind +differential temperature	0.18 in. (4.6 mm)

These were well within the figure of 0.31 in. (8 mm) at 25 m.p.h. which was wanted.

The effect of icing on the structure was not included in these deflections but a half inch of ice adhering to the surface would only increase the R.M.S. values by a very small amount.

Position indicators

When a radio telescope is being used, the operator needs to know in which direction the axis of the instrument is pointing, so some device which reads off the angular rotation of the structure from a given datum must be provided. In the telescopes which have been built so far a central pivot is used to resist the horizontal wind loads. Round this pivot a 360° scale is placed so that with the appropriate reading head the horizontal angular position may be found. In the structure we proposed, the horizontal restraint of the telescope was provided by the horizontal bogies and there was no central pivot. Because a tolerance was required between the bogies and the foundation ring beam, the locus of the position of the centre of the structure, as it rotated, would describe a small circle in plan, with a diameter of half the tolerance gap open at the time.

In this case to find the horizontal rotation of the structure from an initial datum line, it would be necessary to know the apparent angular rotation of each end of a line drawn through the centre of the structure, and it can be shown that the actual angular rotation of the structure would be the average of the apparent rotations of the ends of this line. If two diametrically opposite bogies were taken to be the line, then reading heads attached to them would determine their positions on a suitable circular scale placed on the top of the foundation ring beam and so find the distance they had moved from an initial position. Various devices for the reading heads and the circular scale were looked at and it appeared that the angular position of the reading heads could be found to within .001°.

As these readings only fixed the position of the structure at the level of the bogies, the twist of the structure between the bogies and the dish had to be known. This was not calculated in detail, but appeared to be of the order of .001°. The dynamic response of the structure to wind gust was not derived either, but the

high inertia of the structure would permit close control of its movement during gusty conditions.

To measure the elevational angle, a device which was essentially a pendulum attached to a reference scale on the structure would have been used.

The construction period of such a structure would be sufficiently long for the concrete to be mature before operational loads act on it so creep effects would be small. Some movement due to creep could still occur during the early life of the structure so a system of determining the relative positions of the main beams by optical means was designed. Originally it was proposed that the relative positions should be checked periodically to obtain the effects required to calculate the movement of the outer beams relative to the central beam. However it appeared that it would be easier to continually monitor the relative position of the three beams. The data from this monitoring system would be used by the control computer when calculating the movements required from the jacks supporting the steel girders, to eliminate the effects of concrete creep together with the effects of elastic deflections.

Construction

In the construction method assumed, no unconventional methods were envisaged. Cast in situ concrete was assumed for all concrete members although the cranage envisaged would allow for precasting of the main frames in segments (à la Sydney) if this were advantageous.

The structure would be built with the top of the toroid horizontal, i.e. with the axis of the dish at 45° After constructing the foundation and lining of the pond, the toroidal shell would be built, supported off the ground. This shell was sufficiently strong to allow the rest of the structure to be built off it, so the frame members and the horizontal beams were to be cast using temporary steel trusses supported on the shell.

The steel girders and feed support legs would be erected in large or small prefabricated sections depending on economic considerations and the panels of reflecting surface prefabricated in 40 ft. by 40 ft. (12.2 m x 12.2 m) units.

Conclusions

As a result of our work we showed that, in principle, a concrete structure could be built which met the specification which we were given. In a full scale design study there would inevitably be many problems which would arise but the fundamental problems had been solved, for one of the advantages of our proposal was its basic simplicity. The ease with which the deflection requirements could be met was most noticeable. For example, we took a solid skin over the whole dish, as the wind load could be carried without difficulty, whilst in the specification the use of a mesh to reduce the wind load on the outer part of the dish was allowed.

Reference:

(1) LOVELL, Sir Bernard. Astronomy and the engineer. The Maitland Lecture 1965. *Structural Engineer* **43** (1), pp. 3-10, 1965.