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The Berry Lane Viaduct

Jorgen Nissen

The Berry Lane Viaduct will carry the London North Orbital Trunk Road over a deep valley in Chorleywood. A small country road called Berry Lane runs along the bottom of the valley and the Metropolitan railway runs on the south slope. The valley is full of trees and very pleasant and much used by walkers. It is a residential area and private houses lie in the woods.

The North Orbital Road is being designed by the Department of the Environment, but in April 1971 we were asked to prepare a design for the viaduct. It was only natural that our clients should stress the importance of the environmental aspects in their brief as these were the *raison d'être* for the structure. An embankment had earlier been considered; it would have been some 18 m high and around 100 m wide at the bottom and would completely have destroyed the woods. It would also have severed the valley into two separate parts which would have been connected only by a 100 m long tunnel for Berry Lane. This tunnel would have had to be artificially lit 24 hours a day.

Our main task was easy to formulate: we should design a structure which would not act as a barrier and which would preserve the character of the woods as much as possible. It was more difficult to see how this could be done. A 35 m wide slab suspended some 18 m above ground is an imposing feature which cannot be hidden away, but in this case the woods seemed to help us. Inside the woods there are no long views along the valley; the trees stand in the way, and the ground is mostly in the shade of the trees. The trees are the same height as the viaduct and we felt that we might soften the impact of the structure if we could design a light and unobtrusive support structure of the same scale as the tree trunks. We would, so to speak, try to keep the feel of the wood under the bridge. To succeed in this, it would be very important that the woods should be disturbed as little as possible during construction and that their natural character could easily be restored in the areas that were disturbed, so that a scar would not be left for long. This limited the construction methods that we could consider.

Access to the site is restricted and some construction methods are therefore more attractive than others. Berry Lane passes through narrow tunnels on both sides of the site so that only light traffic could pass this way. The main access should therefore be from the north along the cutting for the future road where it is wide to provide for sliproads. This cutting will, of course, not be available at the beginning of the contract and the foundations will, therefore, most likely have to be constructed from Berry Lane. Fortunately, they can be made as simple footings as we found firm chalk below a very thin layer of alluvium over most of the area.

The structure should also be suited for construction over Berry Lane and, more important, the railway. This is very busy as it carries the Metropolitan underground trains as well as trains for British Rail. Work over the

Fig 1 below
In designing this bridge, we were determined to preserve the woods through which it passes, as much as possible. From this consideration followed ideas for a construction method and appearance and these principles resulted in the proposed tabletop structure. (Photo: McMasters-Christie).

rails can only be carried out at night and then only for three hours at a time and there are restrictions on the methods of construction that the railway authorities will allow.

Having identified these main considerations we were soon able to establish some general principles. It seemed right that a simple repetitive structure should be used; we could see no reason to single out either Berry Lane or the railway and make larger spans over them and we found in our cost estimates that the economic range of spans in any case is larger than both these spans. The conditions for prefabrication are obviously favourable and we thought it would be useful to use a split structure with separate decks for each carriageway. This should give some advantages in construction but, more important, we could make a gap in the wide slab which would otherwise blot out the sky over the whole width of the road. This, we hoped, would help the vegetation under the bridge in establishing itself again as some light and rain would pass through the gap.

At this stage we prepared sketch designs of some structures which all fulfilled the requirements mentioned so far. They range from continuous girders to tabletop structures with precast beams suspended between in situ tabletops. We considered different cross-sections and materials and compared the schemes on their functional, technical, economic and aesthetic merits. In the end we chose a tabletop structure. This structure compared well with the rest on all counts and we thought it was superior in one respect; each tabletop was supported by four very slender columns which could be made nearly as thin as the trees that surround them. Other structures were supported by two columns which were necessarily much thicker. We compared these two column types in simple models and found the thicker columns much more conspicuous although only half the number was used. As mentioned before, the trees tend to hide the supports if they are slender enough. We also found that the relationship between the four columns is important; they should be close enough together to read as a group and not just appear as a forest, but they should not be so close that the light and diffuse character is lost.

Tabletop structures form a clear and simple structural system and this perhaps adds to their appearance. It is easy to see how the structure carries its load and how it is constructed. All the precast spans rest simply supported on the tabletops and each tabletop is in itself a stable structure which carries all loads that act on it, including bending and horizontal loads, to its foundation. We chose to make the tabletops comparatively short and to support the suspended spans on hinges at both ends in order to limit the bending that has to be carried to the foundations.

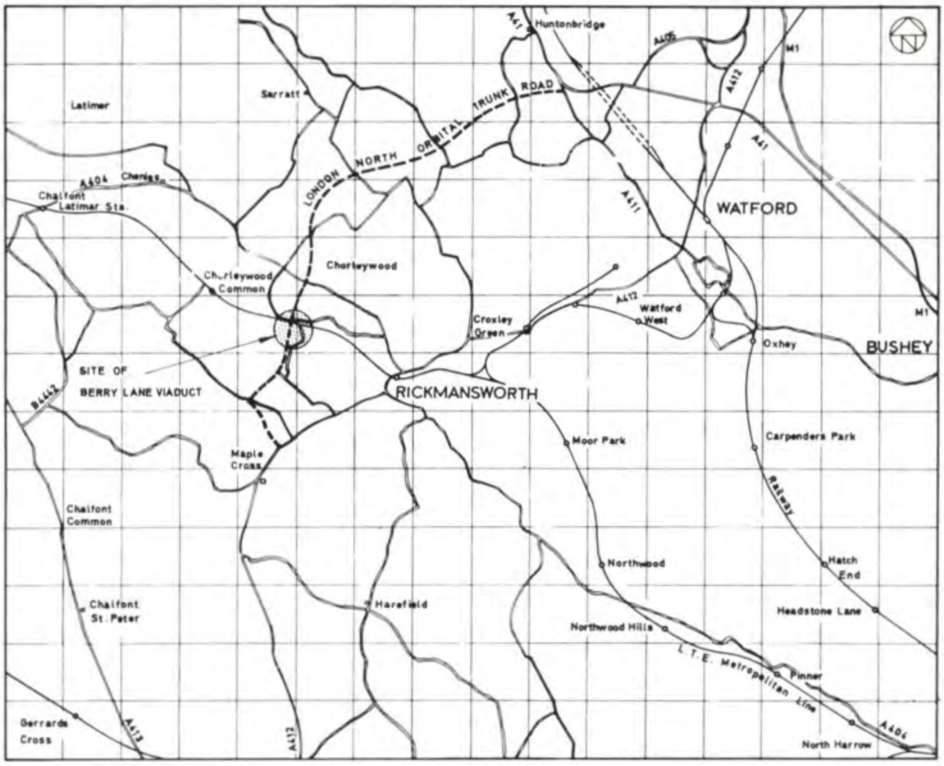


Fig 2
The whole contract covers the North Orbital Road between Hunton Bridge and Maple Cross.

Fig 3 below
The main access should be from the north along the cutting for the future road where it is wide to provide for sliproads.

The construction of the tabletop structure is well suited to this site. The smaller part of the structure is cast in situ; it is reinforced only, not prestressed, and we have made the members bulky so that the concrete can be easily placed. Each tabletop is stable at all stages during its construction and it provides a platform from which to place the precast beams. These can be brought in from one end. There are no bearings or complicated joints within the in situ work to slow up construction and scaffolding can easily be erected on top of the footings. We thought at first that it would be a good idea to save some of the scaffolding and we studied a scheme for casting the tabletops at ground level and then jacking them up on the columns. This produced no savings and it introduced an awkward joint to be made when the tabletop had reached the top, so we gave up the idea.

Having taken the fundamental decisions about the structural system and how to build the

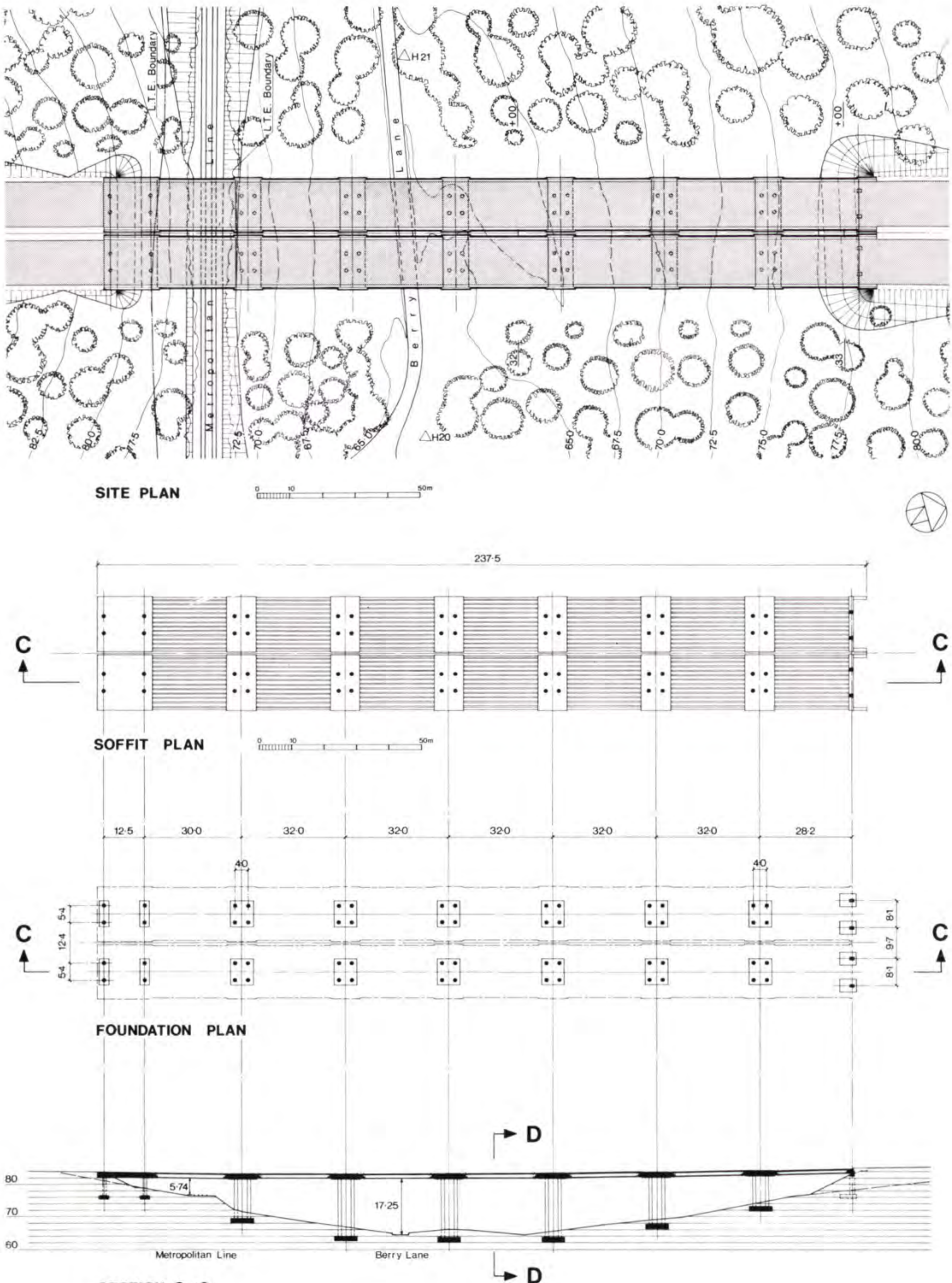


Fig 4
Site plan, soffit plan, foundation plan, section C-C.

bridge, the deck structure needed to be refined. We felt especially that our sketch designs for the suspended spans could be improved upon. They were based on standard systems of precast beams but we thought that we could choose from a wider range as we should need enough beams to make it economically viable to build special shutters. We were also not entirely satisfied with the appearance of the standard beams in this particular case. This bridge will not be seen at a distance, but experienced mainly by those driving or walking under it and the shape of the soffit is for that reason particularly important.

We therefore made a special study of decks made up from precast beams with in situ topping. The criteria are obviously that the precast component should be easy to make, transport and erect and that the in situ component should be easy to shutter and make. The choice of any system, however, will be greatly influenced by its ability to distribute the effect of concentrated loads over a wide part of the cross-section. Most bridges in this country have to be checked for a load of 180 tons which is concentrated on a small area, and it is important for economy that this large load should be carried by a much wider part of the deck than the one on which it acts. Our study resulted in a precast trough unit with an in situ slab on top. It compared well with standard systems in our cost estimates and we thought that its advantages, particularly in construction, might well in the end make it the cheaper deck. The beams can be launched from the northern end onto simple laminated rubber bearings, and they are stable and form a complete platform as soon as they are placed, which is important over the railway. There is no prestressing to be done on site and the reinforcement in the top slab is very simple. Shuttering is not needed between the beams and easily supported over them. There are no transverse beams except at the ends, where they are easily cast, but the final structure is nevertheless very good at distributing loads transversely because of the high torsional stiffness of the boxes.

We produced a very refined design for the trough units as they are to be cast under factory conditions and because each superfluous bit of material would be repeated 84 times. To make sure that the thin webs could be concreted without problems we had a test made. We had a 3 m long specimen cast complete with reinforcement and the cable ducts in their worst position. The test confirmed our design and we shall be able to provide tenderers with more information as well as be able to answer questions with the confidence that results from having cast one already. The test also gave us a chance for a preview of the concrete surface that we are planning.

We studied many complicated structures for the tabletops, but chose in the end a solid reinforced slab. A solution for the abutments practically presented itself. The junction between earth embankment and structure is very important, but often difficult to design. Here we could make the tabletops longer and bury the hind legs in the embankments. The trough units need not be adjusted here and all precast beams can be made identical.

We presented the scheme to the Royal Fine Art Commission in November 1971 and are now (May 1972) in the final stages of design. We have estimated the cost of the project to be £745,000 or £93 per m². The viaduct is only a small part of the whole contract which covers the North Orbital Road between Hunton Bridge and Maple Cross. The Hertfordshire County Council Sub Unit of the Eastern Road Construction Unit are designing the rest of the works. We hope that tenders will be invited in late autumn; work should start on site early next year and last two years.

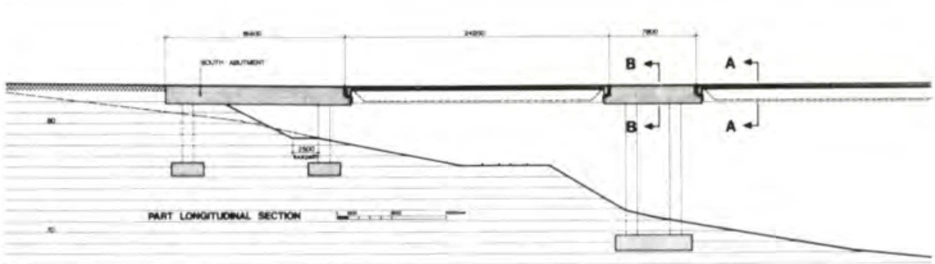


Fig 5
The south abutment is a long tabletop with the back legs buried in the embankment.

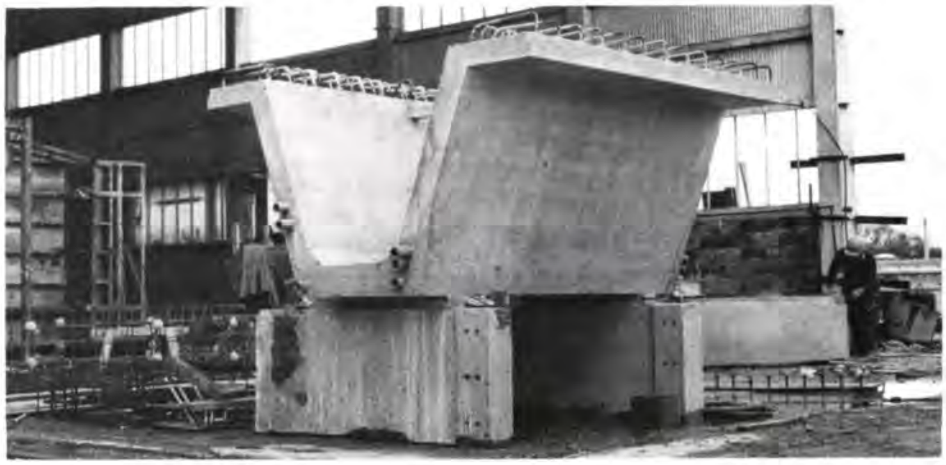


Fig 6
We had a test specimen cast to confirm our design.

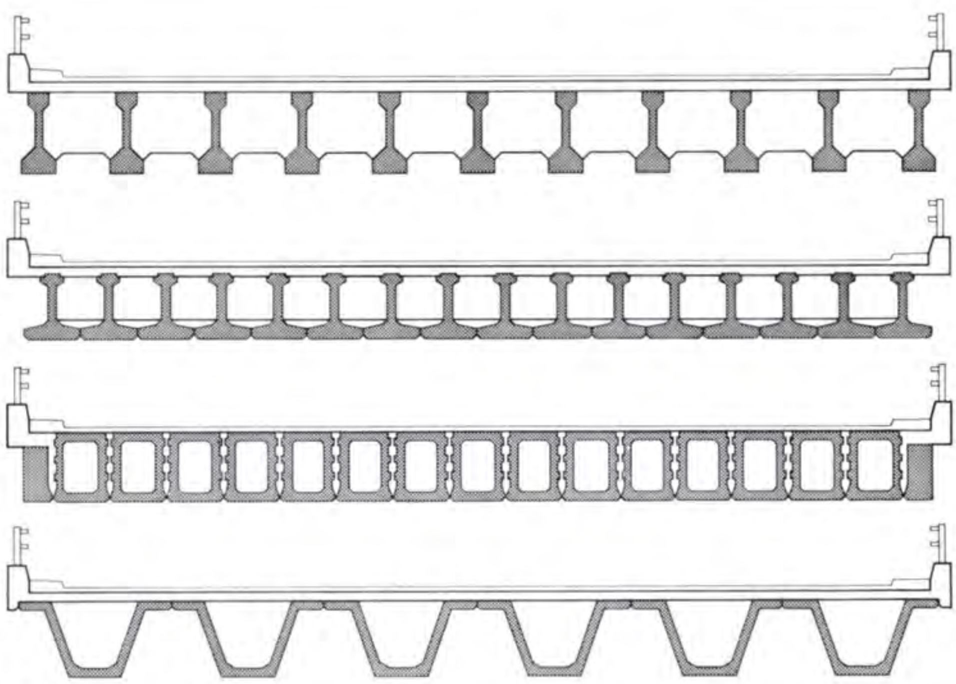


Fig. 7
Some of the structures for the suspended spans that were compared.
The three upper structures are based on standard precast beams;
the lower is the design that was chosen. Precast beams are shown shaded.

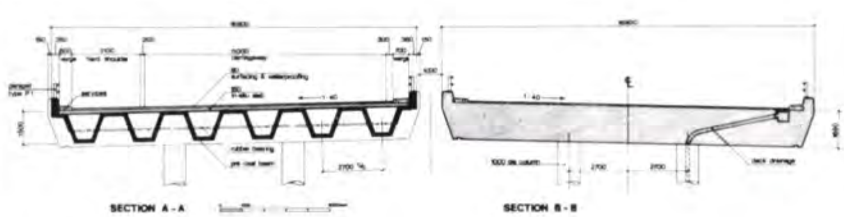


Fig 8
Cross-sections. The suspended spans (to the left) are partly precast,
the tabletops (to the right) are cast in situ.

The Evry 1 Competition

Michael Barclay

The 1965 development plan for the Paris region is arguably the most extreme application of a professional philosophy which is only now beginning to lose its hold on city planners round the world. The basis for the plan was the proposition that half the population of France would live in Paris by the year 2000 and that Paris should accordingly be planned as a city of 50 m. people who would be highly mobile and materially rich.

The ancient city, still separately administered within the boundaries of its ramparts which now roughly coincide with the *boulevard périphérique* motorway, would remain an increasingly depopulated commercial and cultural centre for the new city region which would stretch 400 km along the Seine from Fontainebleau to the sea, absorbing Rouen, Le Havre and many other towns in the process.

The new motorways and railways to serve and link such a conurbation have yet to be defined, let alone constructed. Meanwhile, the present Paris suburban region – *the Banlieu* – is exploding with every kind of housing and commercial development and the small communes which surround the city are being overwhelmed by a largely unplanned and unbeautiful growth.

It is in this context that the French government set up the mixed-economy *Établissement Public* for the Défense area on the northwest edge of the city a decade ago. La Défense has already become the business centre of Paris and it will soon be one of the city's densest residential areas. It can be seen as the first local city centre of the future metropolis with its express metro and motorway links already established.

It is in this context also that the first five new towns of the Paris region have been designated – Cergy-Pontoise, Bry-sur-Marne, Mélun-Sénart, St. Quentin-en-Yvelines and Evry. Each lies within 40 km of the centre of Paris and each should serve a population of the order of 500,000. (Fig. 1).

Evry: the first stage

The new towns have similar administrative responsibilities and powers, (not unlike those of a British new town development corporation but with less democratic control) and similar briefs, but they have little else in common. Each professional team has taken its own view of the possibilities open to it and of the constraints it should observe. Each town will develop in its own way. We became involved in the first stage of Evry's development and this is the one which is described here.

Unlike most British new towns, Evry is planted astride an agglomeration of existing small towns and housing estates which include some of the fastest-growing urban areas of the region. Evry has been made the headquarters of the new Essonne Department and the *préfecture* was its first building.

The centre of the new town will be built on 450 hectares of flat open land between the A6 (*Autoroute du Sud*) and the old N7 in a bend of the Seine about 15 km south east of Orly airport. It will eventually be served by a branch of the main line railway and by the new city motorway system along the southern side of the Seine valley. The forest of Senart is just across the river to the east and the old town of Corbeil is one of the member communes to the south.

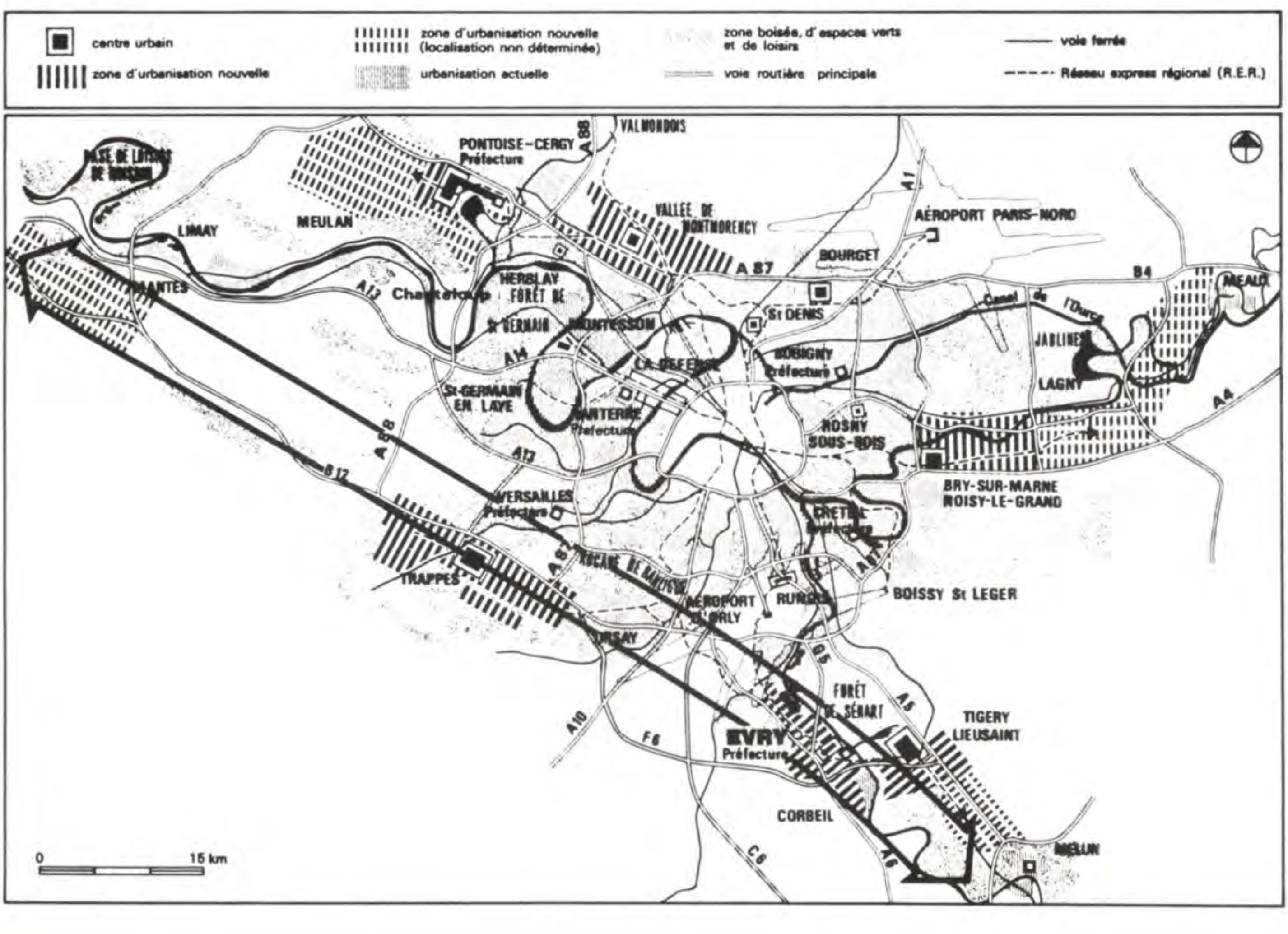
The *Établissement Public* for Evry new town – EPEVRY for short – decided to build this centre in five parts. The 'heart' will contain the main shopping complex, the main entertainment and cultural buildings, and the administration. The four central sectors, radiating from the heart, will contain a total of 20,000 dwellings together with their schools, local services and open spaces. Each sector will be built over and around a radial line of the new public transport system and include a central pedestrian route. Vehicle and railway access to the town centre will be along partly sunken routes through the parkland between the four central sectors and the distributors will feed into the residential areas in a Radburn pattern on a grand scale.

It is essential to the whole conception that the new town centre should become the effective focus, not only for the four new central sectors but for the whole conurbation of 14 communes which will eventually house the half million population. The new public transport system, initially conventional buses on their own track, is intended to be one of the principal means of achieving this aim.

There are some remarkable assumptions and contradictions implied in the original brief which became apparent as we worked on the first sector, but it is first necessary to describe the competition, which is still in progress as this is written, and to explain our participation in it.

In April 1971 EPEVRY invited suitably qualified groups to enter a competition for the design and construction of the first central sector – Evry 1. The competition was in two

Fig. 1
Map of region.



parts – an eliminating round ending in November with the selection of four finalists, and the final round ending in the summer of 1972 with the selection of the winning group. There would be consolation prizes but no first prize – only the immediate award to the winners of a £60m. contract to finance and build Evry 1 in five and a half years flat.

We were invited by Denis Sloan, a French architect with whom we had not worked before, to join him as civil and structural engineers in forming the core of a design team to make an entry. He had himself been invited by a French developer, SMCI, who was forming a promotion team to complete the group. We accepted and the group was later joined by OGER the contractors, and by three large banks, as well as by a number of professional consultants including Henri Trezzini, Maurice Khayat, and Shankland, Cox & Associates.

Evry 1 covers 82 hectares and includes 7,000 dwellings, 11,000 parking places, over 40,000 m² of shopping and offices, 24 schools, about 60 other public buildings, and 26 hectares of parks and gardens. It also includes many kilometres of roads and public transport routes, as well as bus stations, cycle and pedestrian ways, and a glorified permanent Motor Show.

Briefing

The briefing documents were of two kinds: the philosophical and the technical. While the latter were meticulously explicit, the former were poetically abstract. An enormous professional, commercial, and building enterprise had to satisfy God and Mammon according to their different laws. It was rather as if the GLC's Hook New Town report of 1963 had been simultaneously translated into a development dossier by Trafalgar House and an article for *Architectural Review* by Lewis Mumford. This may partly explain the interesting variety of solutions offered, not all of which were academically coherent.

In presenting our own proposals, Denis Sloan pointed out some of the difficulties inherent in the brief. Evry 1 had to be quiet and peaceful but was surrounded by motorways and other roads of such size that the external traffic noise could be deafening across the whole area for much of the day. Evry 1 had to be a self-sufficient community but adequate shopping was rigorously excluded in order to strengthen the regional centre and few jobs could be provided by the trivial service activities allowed. Evry 1 had to have a lively and visually enclosed pedestrian mall from end to end, offering a variety of views and opportunities for social exchange, but the huge and normally deserted track of the public bus route had to follow almost the same line and no pedestrians were allowed to step onto it. These and other inconsistencies were demonstrated by superimposing the existing street plan of Paris from St. Germain des Prés to Place St. Michel on the main axis of Evry 1. The brief for the new town demanded all the social and environmental qualities of this ancient city quarter with its narrow and intricate streets, hundreds of shops and restaurants, and teeming population, but without any traffic nuisance; and the same brief nevertheless precluded many of the elements which make Paris so attractive in spite of its overcrowding and pollution.

A simple and economical plan

Our own team decided not to design a regiment of ziggurats over a vast parking basement as some of the sketches in the brief suggested. We went for a much simpler and more economical plan: simpler in being more like an old-fashioned town in which people can easily make themselves at home, and because the test of success would be how much the new town was enjoyed by its citizens: economical in avoiding costly

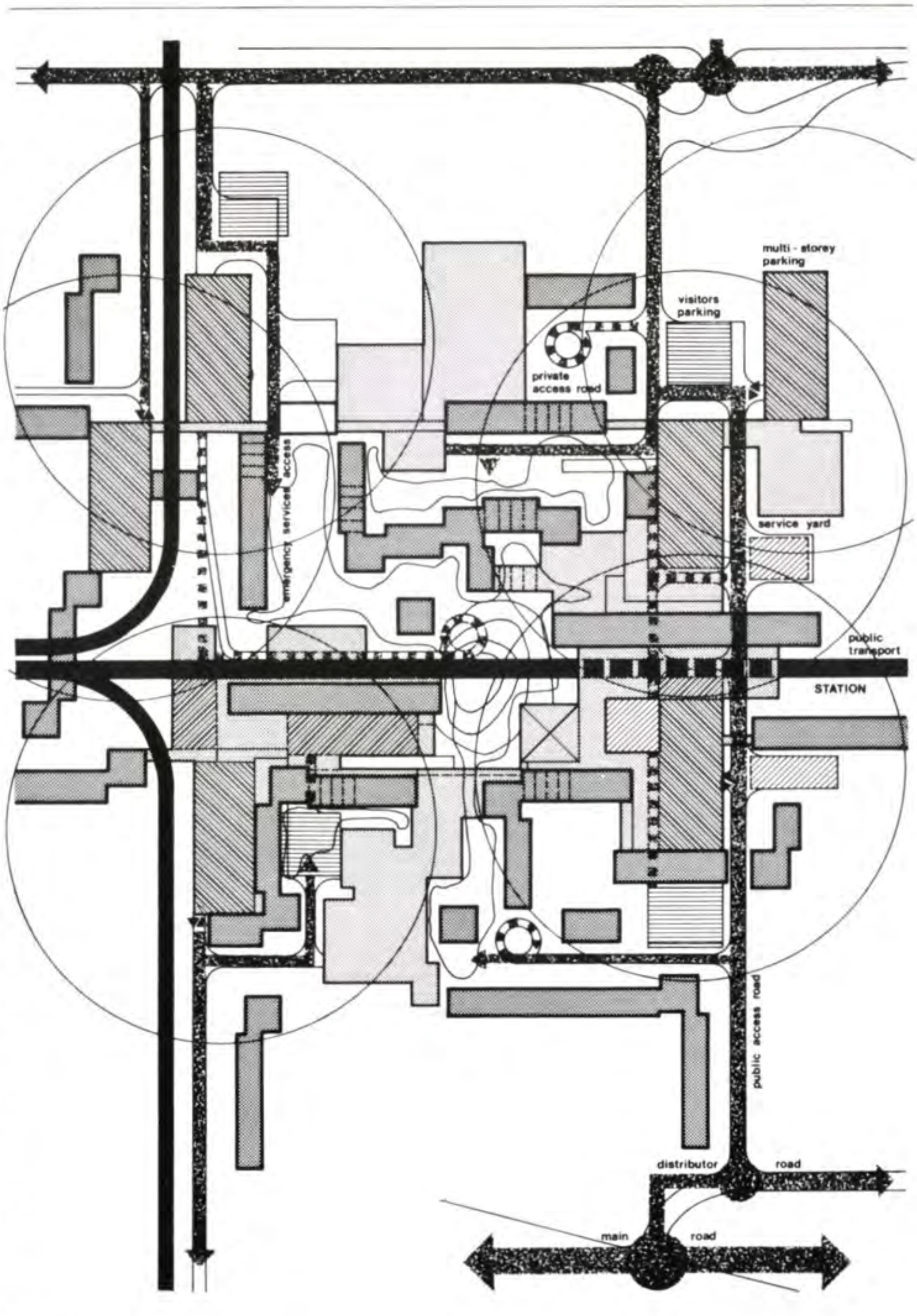


Fig. 2
Access and transportion plan.

structures and minimizing roadworks, and because only the economical would eventually be built regardless of what scheme won the competition. Moreover, only an essentially economical plan could afford good landscaping and materials. If we were really to help in the search for an acceptable form for future cities, our solution had to meet the needs of local authorities, promoters, and citizens alike.

We attempted to create a town of medium-rise buildings arranged in a complex pattern of courts rather than streets, so that pedestrians and cyclists could move freely at ground level in many directions, while vehicles could only come to load and unload, parking being provided in blocks on the approach roads. This largely avoided underground parking and the dismal landscape of housing estate roads while giving the best possible opportunity of planting trees and gardens in undisturbed topsoil. Some of the devices used, such as bollarded vehicle lanes across pedestrian courtyards, were unorthodox but based on successful experience elsewhere in France. (Fig. 2).

We made a case for more space to be reserved for local shops and services which could be built as the demand developed, and we planned to allow for other infill and extension building over the years. We investigated district heating by exploiting the deep natural hot water reservoirs below Evry and we proposed a unified structure of linked squares for primary schools and artisans' workshops. But the key to our planning was the modular housing grid developed by Bryan Seymour which allowed a very large number of dwellings of different types to be fitted into an extremely simple in situ concrete egg crate frame which could go round corners and change width or span without difficulty. The same economical structure made the honeycomb for many ancillary uses as well.

Although we were not among the finalists we are continuing to develop our ideas with the other members of our group and the following article describes the modular housing system itself.

Acknowledgement

We should like to acknowledge the valuable contribution made by David Bishop who wrote the report on transportation and parking.

Mass housing - flexibility with standardization

Bryan Seymour

Introduction

We have, over the last 10 years or so, become increasingly involved in the preplanning of our housing projects. The structural implications of modern building methods require not only our early involvement in the design stage, but also our full appreciation of many other problems that are not purely structural engineering.

Our recent exercise at EVRY serves very well as an illustration of some of our thoughts on the problems of mass housing. Although most of the problems and solutions shown are international, it must be remembered that these plan types were developed for a particular context in France.

The problem

The new community of EVRY required 7,000 dwellings and they all had to be built in a relatively short time. These dwellings ranged from one living room or bedsitters, to full family, six-roomed dwellings. In addition to this basic range of six types, each type had to be available in five grades from very simple dwellings for the tenants with a maximum subsidy to luxury dwellings for owner occupiers. There were only three grades of dwelling size, the changes in fixtures and fittings and so forth providing the other grades. A four-living room dwelling, for example, was available at 77, 82 and 86 m². It is difficult to relate this directly to our own housing standards as we rate a dwelling by the number of bed spaces and not living rooms. Also there is the problem of service ducts and stores and similar spaces, either being part of, or in addition to, the basic area. The actual dwelling areas were only part of a whole series of areas that included nett and gross area ratios for access and service facilities; also, the dwelling densities varied in different parts of the project.

The majority of dwellings for EVRY were to be provided as single-level apartments in various types of block with internal corridor access. These blocks would vary from low walk-ups to high-rise point blocks.

To set the background to this planning exercise, the following summary embodies most of the requirements that we thought needed to be satisfied to provide successful mass housing.

Each particular grade of apartment must be available in a variety of plan forms, with dual aspect for the larger dwellings. These apartments must be able to combine to form all possible cluster arrangements and, where necessary, steps and staggers must be incorporated to clearly define different dwellings, both externally and internally.

All apartments should have a large, safe, usable balcony in the form of a semi-enclosed loggia that is an extension of the living space, and a clearly identifiable entrance with a recess in the porch.

Internally the living areas should be as flexible as possible, and capable of providing a large combined area when necessary. Family dwellings should have sleeping and bathroom facilities separated from the remaining living areas. The entrance and hall should give easy access to all parts of the dwelling.

The kitchen should have dining and laundry spaces and, in the bedsitter, the laundry space can be provided in the bathroom. All dwellings should have as much storage space as possible.

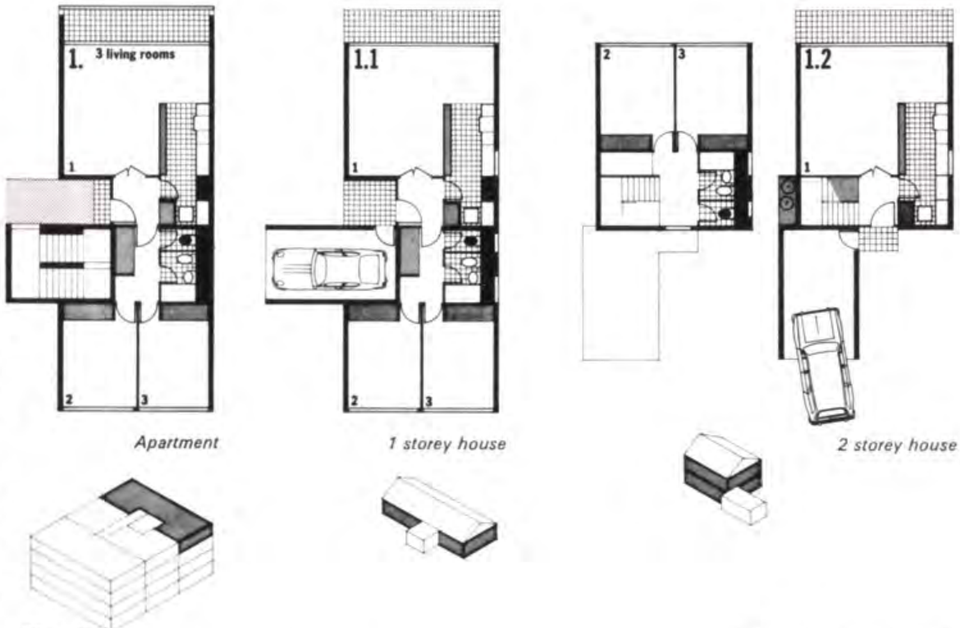


Fig. 1
Flexibility of dwelling arrangement

The same basic dwelling arranged 3 different ways

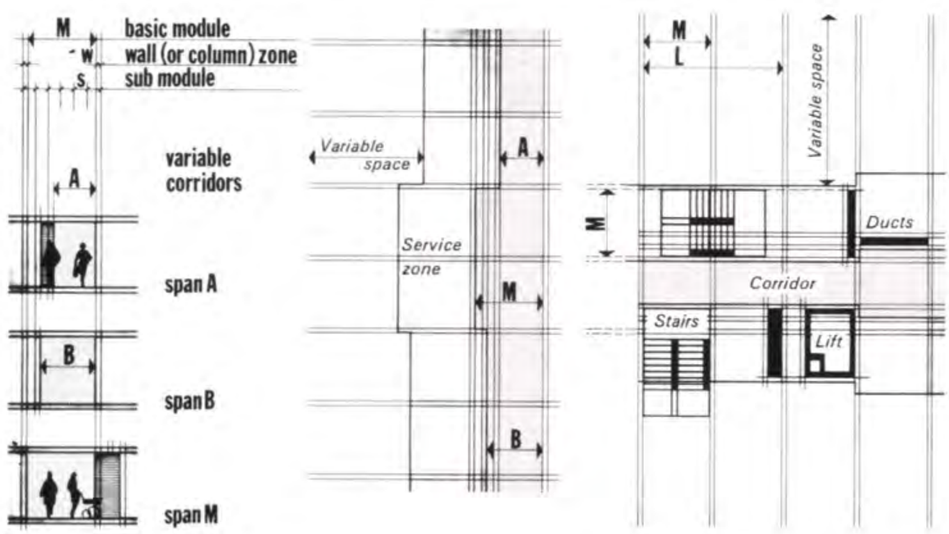
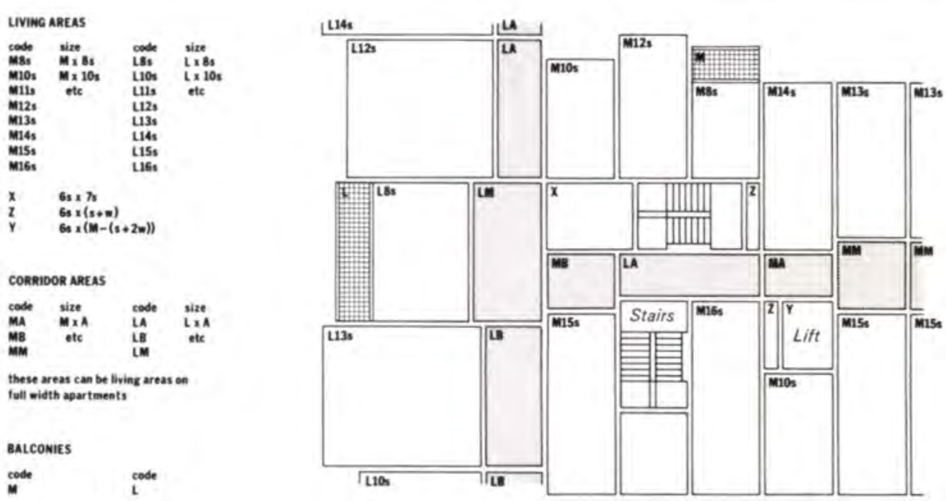


Fig. 2
Basic planning grid

This exercise shows the inter-relationship of space and structural zones. The actual sizes are irrelevant in an overall sense



$M+2w=6s$

for dimensions see BASIC PLANNING GRID

These areas could be precast but not necessarily so; only some of the wall zones are required

Fig. 3
Area co-ordination

A solution

With the available time and resources the dwellings had to be standardized, and whenever possible, system and industrialised building methods would be used. This standardization however, had to be capable of producing variety and not the usual monotonous rows of repetitive blocks.

The first set of illustrations in Fig. 1 shows simply how three types of dwelling; an apartment, a single storey house and a two storey house, can all be planned within the same basic dimensional framework. The following exercise concentrates on the apartment form, as this comprised the vast majority of dwellings to be provided.

All of the apartments serve the same purpose and are made up of the same basic parts. Some of these parts, such as the bathroom and kitchen, are very complicated, whilst others, such as bedrooms, are not; they are just space. Similarly, with the communal areas, the lifts and staircases are complicated but the corridors are again just space.

The basic planning grid, Fig. 2, highlights the different areas of complexity and shows the location of the variable space. The service zone is the space where, for example, the bathroom unit and lift will be located. The basic module (clear wall-to-wall dimension) M positions the wall zones. Not all of the wall zones need to be used, and in the main the subsequent plans use a double M unit. When a double M unit is used, it must be added to the dimension of the wall zone to give dimension L. For instance, if the standard wall-to-wall dimension M is 2.7 m and the thickness is 0.2 m the wall-to-wall dimension L is (twice times 2.7 plus 0.2) equal to 5.6 m. It is, of course, possible to have various dimensions for M in any one project, but the number of permutations rapidly escalates.

The corridor spans have been restricted to three basic dimensions; the original M plus A and B. An important feature to note is the facility to arrange the wall zones so that the corridor can turn a corner.

The variable space either side of the service zone gives most of the basic flexibility, but even here it was possible to rationalize so that the increase in depth of this variable space is in steps of a sub-module. Fig. 3, area co-ordination, illustrates the full range of areas or parts necessary for the EVRY dwellings.

The coding of these parts is a simple two-dimensional code for its size. For example a corridor part 'LA' is L long by A span and, similarly, if s (sub-module) is 0.5 m and M is 2.7, part M10s is 2.7 m span by 5.0 m depth.

Two particular sets of parts are shown in Fig. 9, and, from these parts, apartments 8, 8.1 and 8.2 (in Fig. 8) can be planned. The bedsitter dwelling is shown in two versions, one of which allows for a staircase or lift insert.

The various living patterns shown in Fig. 4, together with the kitchen and bathroom sheets, Figs. 5 and 6, were used to evolve the basic grid. These living patterns show how the particular plan forms accommodate the family's changing requirements. The standard kitchen in Fig. 5 is in fact only standard for the complicated cabinets; the additional space and worktops are uncomplicated and so can be very flexible. The standard bathroom is a contained unit and only needs to be variable for the small dwelling when the internal partition is removed. This is for two reasons: firstly the restricted area of a small dwelling necessitates the use of one door only and, secondly, the extra space created provides a washing machine or laundry space.

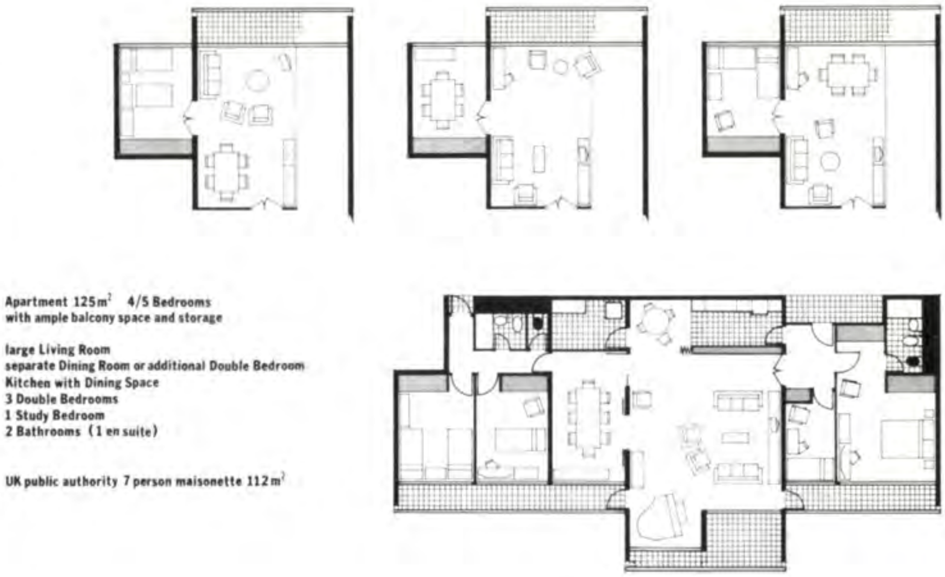


Fig. 4
Variable living patterns

Fig. 5
Standard kitchen

Kitchen variations are made by adding simple worktops and space

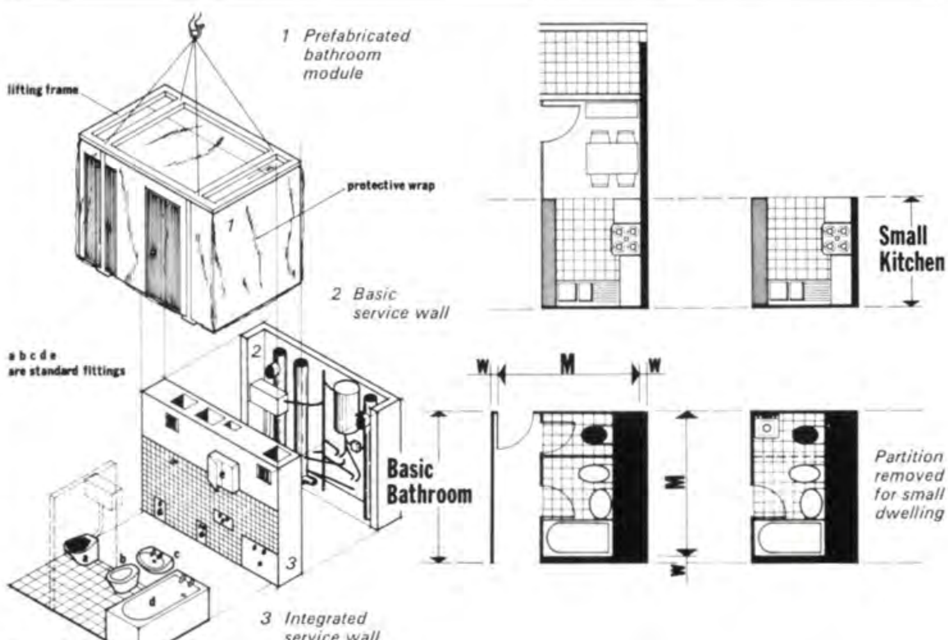


Fig. 6
Small kitchen and standard bathroom

The dwelling arrangements in Figs. 7, 8, 10 and 11 all show possible apartment layouts that conform to the basic grid, and, in addition to these, many more can be evolved. As previously noted, the structural zone is only used when necessary and, in the majority of cases, a double span slab has been used. Fig. 12 shows how blocks can be increased in size and how the various dwellings are combined to form continuous blocks.

The photographs of the architect's model clearly show how the blocks of apartments were to be articulated and varied in order to be totally integrated into many different situations.

The drawings were prepared by Bryan Seymour and David Lay.

Fig. 7
Dwelling arrangements

Note:
Each variant can be to the opposite hand

Balcony variation

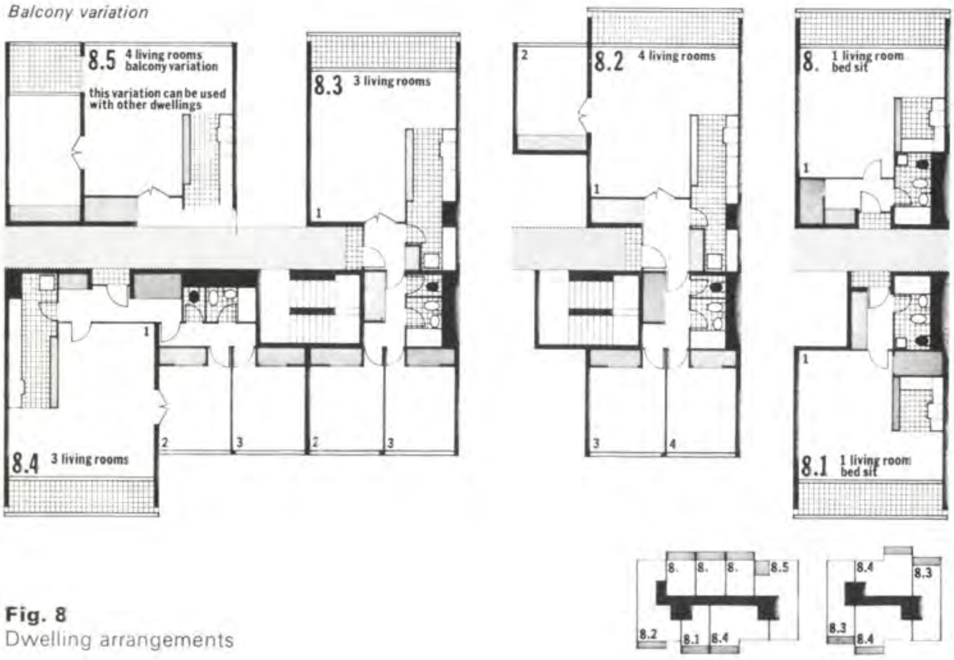
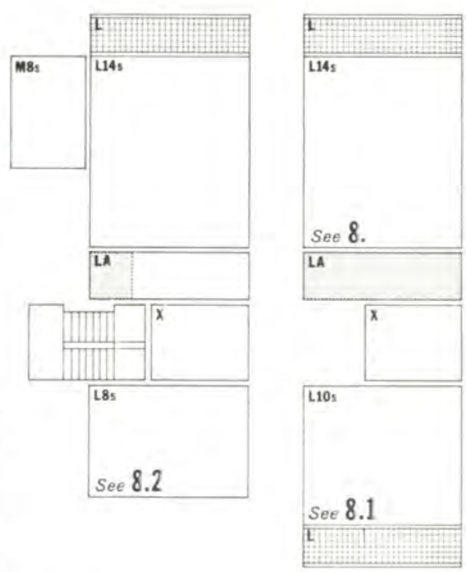


Fig. 8
Dwelling arrangements



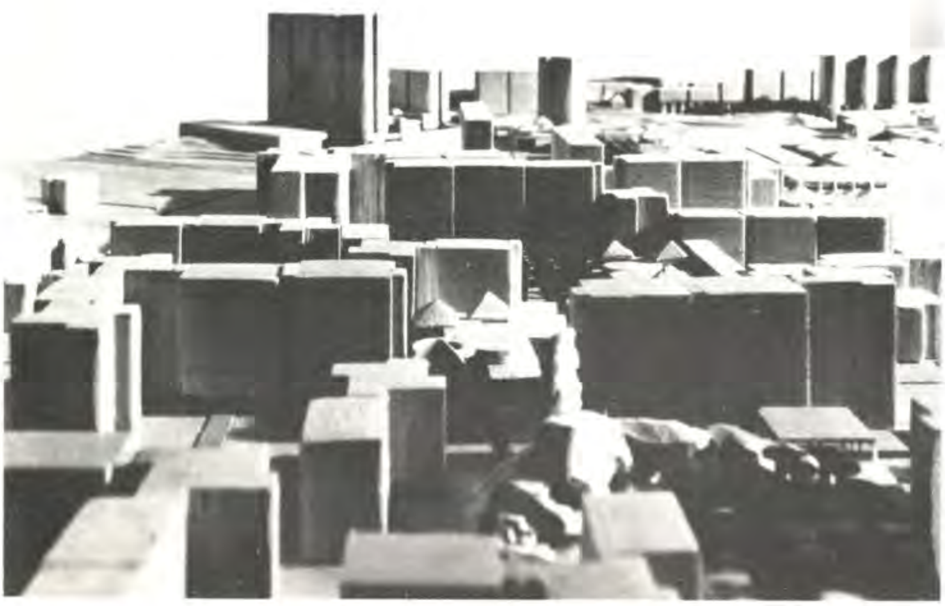
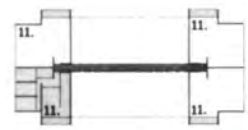


Fig. 9
Area co-ordination
for typical dwellings

Point cluster used for linear continuity

Fig. 10
Dwelling arrangements

Cluster variations



End of corridor unit

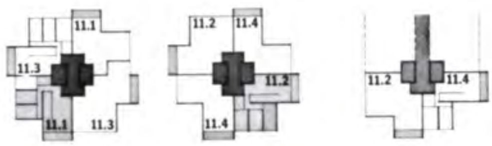
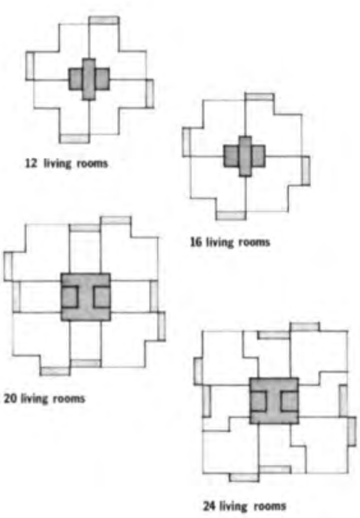
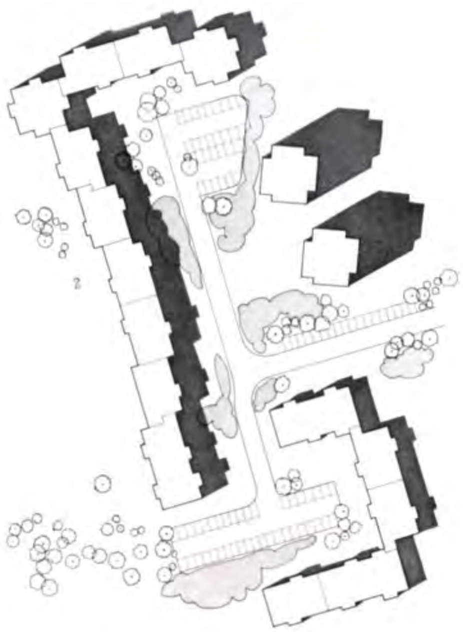


Fig. 11
Dwelling arrangements



An example of cluster development



Some dwelling integration possibilities

Fig. 12
Blocks can be increased in size and combined to form continuous blocks

Recessed porches

Articulated corridor with individual entrances

Fig. 13
Corridors

A study of a large landslide at Herne Bay, Kent

Ted Bromhead

This paper won the 1972 Cooling Prize of the British Geotechnical Society.

Stratigraphy

The East Cliffs at Herne Bay run for several miles in an east to west direction, rising to a maximum height of 38 m OD, and are composed of the lower beds of the London Clay. A weakly cemented sandstone, the Oldhaven Beds, underlies the London Clay and their interface dips slightly from east to west, allowing the sands to form steep cliffs in the east, above which the London Clay thins out, and lies at depth to the west. The clay is weathered brown to a depth of approximately 13 m, and at a greater depth the colour changes again from a grey-brown to a bright grey-blue. In general, the penetration of weathering from the cliff face is negligible.

Occurrence of large landslides

These cliffs have a history of erosion and have been subject to considerable study and stabilization work[3] in the past. Generally they degrade by a process of mudflow formation, but from time to time deep seated failures have occurred. Such failures have always occurred in stretches of cliff capped by sand and gravel ('Head') deposits. It has been held that these comprise a reservoir for water, although an alternative explanation may be that they do not allow mudflow activity to any great extent and this causes oversteepening of the cliff by toe erosion. The elevation of the underlying sandstone appears to control the degree of non-circularity of the failure surfaces of such slips.

Position of Miramar Landslide

For a length of approximately 100 m, coinciding with the outcrop of the Oldhaven Beds on the foreshore, the cliffs have failed twice in recorded history and the evidence shows certain similarities in the behaviour of both slides. A brief summary of the information apparent from the Ordnance Survey maps will serve to emphasize these. Fig. 1 is a site plan based on these maps.

Landslide of 1883

The first edition of the Ordnance Survey shows no trace of any failure but, between then and the 2nd series in 1896, major changes had taken place, with local history placing a failure in 1883. About 20 m of the cliff top slid downwards and a ridge and graben structure was visible, similar to that of the second failure discussed below. In the years between 1896 and 1906 the ridge moved forward some 12 m and showed a marked preferential movement at its western end. By 1933 (4th series) the ridge was so eroded that it was no longer identifiable.

Landslide of 1953

Early in the morning of 4 February 1953 another major landslide took place involving the same stretch of cliff. This time approximately 25 m width of cliff top was lost and photographs (Fig. 7) taken after the failure reveal this slipping down between sharply dipping failure surfaces to form a near horizontal graben, while the original cliff face was pushed forward intact to form a sharp crested ridge. The forward movement of this block considerably disturbed the debris at the toe (remnants of the 1883 slide). By 1956 the crest of the ridge had moved forward 10 m, once again showing preference for movement at the westerly end, and a further 15 m of movement took place in the following decade. Two large mudflows formed and emerged around each end of the intact block.

Initial studies

An initial examination of this landslide (called the Miramar Landslide, after a nearby hotel, part of the grounds of which were lost) by Hutchinson [5, 6, 7] revealed that the landslide had taken place on a highly non-circular failure surface and an estimate of the displacement was obtained by a study of the microfossil succession[7]. In this study, the three major landslides in these cliffs were investigated in a similar, although more exhaustive fashion, but it was unfortunate that two of these had already been partially regraded, and the true pore pressure distribution before these works were carried out was impossible to judge. However, an estimate could be made, and it was thought worthwhile to attempt analyses on earlier profiles in order to provide some field data on the behaviour of the residual strength parameter for the London Clay at comparatively high effective stress levels, since the existing data by James[8] was for very small slips in railway cuttings at low effective stresses.

Site investigation

Clusters of borings were placed systematically down selected section lines through both slipped and intact clay slopes. A visual examination of continuous undisturbed samples throughout the depth of the first boring at each location enabled features of interest such as failure surfaces (Fig. 8) to be located, and several piezometers were installed at each location. Flow nets were drawn from the piezometer readings and well back from the edge of the cliff it was shown that the flow was near vertical. Both from the flow nets for undisturbed sections, and by a visual examination of the samples, it was evident that the vertical permeability in the lowest 6 m of the

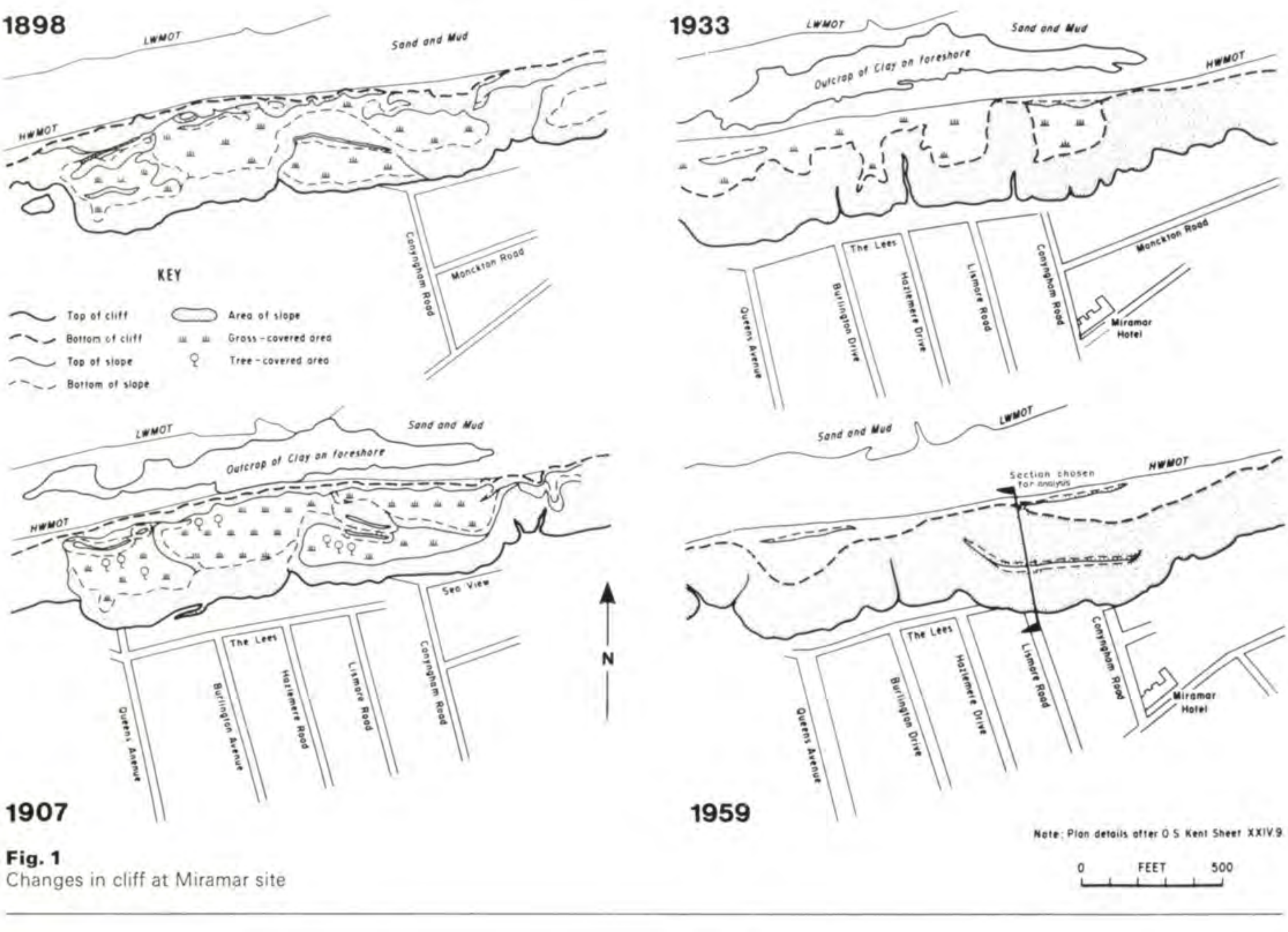


Fig. 1
Changes in cliff at Miramar site

undisturbed clay was substantially lower than in the overlying material, but the presence of sandy and silty layers probably greatly increased the horizontal permeability. It was also logical that the highly disturbed landslide debris should also be comparatively very permeable. A model of pore pressures in the landslide debris being very high, but drawn down with a high hydraulic gradient into the underlying sands, corresponded closely to the observed pressures and was therefore very reasonable, and the pore pressures acting along the failure surface were much higher than the tidally controlled pressures observed in the sands below.

In view of these observations, it was possible to infer that the distribution of pore pressures was similar in 1966 to that observed after regrading in 1970, and that the pore pressures existing at the earlier dates would have been at least as high, if not higher, caused by the percolation of water from the surface down open joints in the slipped mass and undrained loading[11] by falls from the rear scarp. The pore pressures under the 'intact' ridge block were probably kept up by flow through the horizontal sandy layers. One effect of these high pressures, caused by seepage from above, was to prevent the tidal fluctuations of pressure in the sands below from having any effect on the stability. The situation with the tide at its lowest ebb was the most critical and has been taken for analysis.

Surveys

A very accurate survey of the ground profile was carried out in 1966 and further surveys of a section chosen in the westerly half of the slip were carried out from RAF photographs dated 1950 and 1956. (This section differs slightly from that of Hutchinson[6,7]). Taking the 1950 profile as a close approximation to the profile at failure, it was possible to reconstruct the slip surface and Fig. 2 shows both this and the inferred geometry soon after failure. In contrast to the negligible effect of tides on the pore pressures after failure, the stability of the Miramar slip in 1953 was jeopardized by the tidal surge a few days before in two ways; both by increased toe erosion and by a marked increase in the pore pressures in the Oldhaven Beds.

Movements of slipped mass

Continuation of movement was ensured, both by undrained loading (falls from the rear scarp[11]) and by continued erosion and cross sections based on the ground profile in 1956 and 1966. These are shown in Figs. 3 and 4. It appears likely that the ridge moved forward again in the period 1966-1970 when the investigations took place, and the magnitude of this movement is estimated to be about 5 m. Fig. 5 shows how the crest has moved during this period. A total movement of about 30 m is inferred. The micro-palaeontology[4,7] showed that the graben had suffered a vertical displacement of at least 17.4 m.

Initial analysis

The back analysis technique for obtaining shear strength parameters is well established and to complement James' data[8] (which is probably the most comprehensive to date although suffering from a paucity of field observation and dealing only with relatively shallow slides) all of the available slides on the Herne Bay coastline were analysed to obtain the residual, and, in the case of the Miramar landslide, peak shear strength parameters. The Kenney[9] method (Program OA800) was used, at first giving satisfactory results, but it did not give a convergent solution when applied to the 1966 profile of the Miramar slide.

Further analyses

Further results were then obtained from a computer program written by the author using the Morgenstern and Price procedure[10] (Program OA808). Four cases were analysed

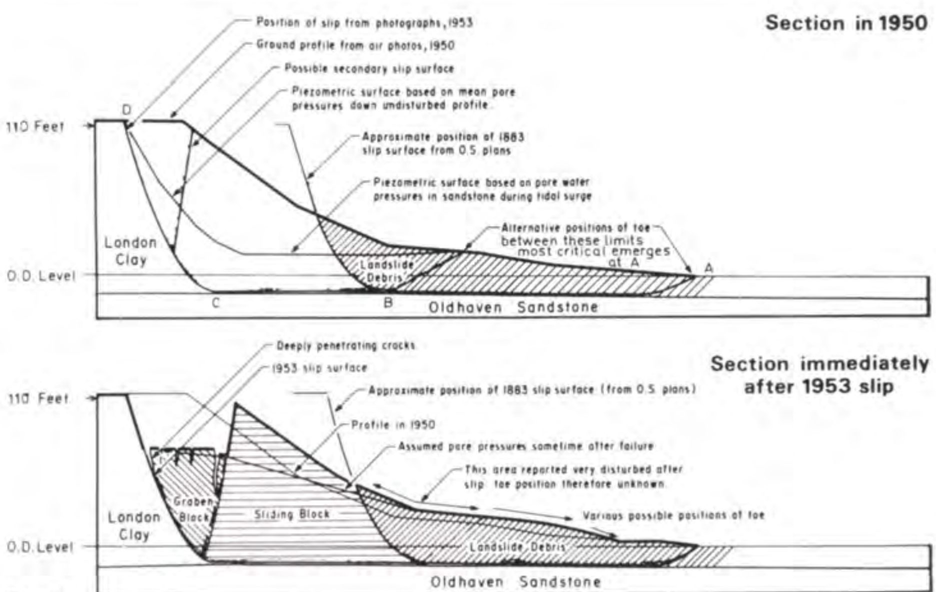


Fig. 2
Section in 1950 and after 1953 slip

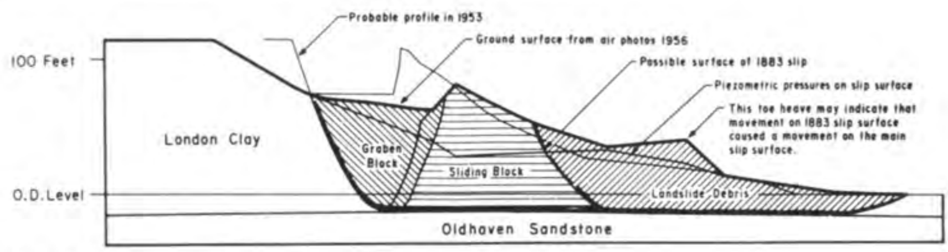


Fig. 3
Section in 1956

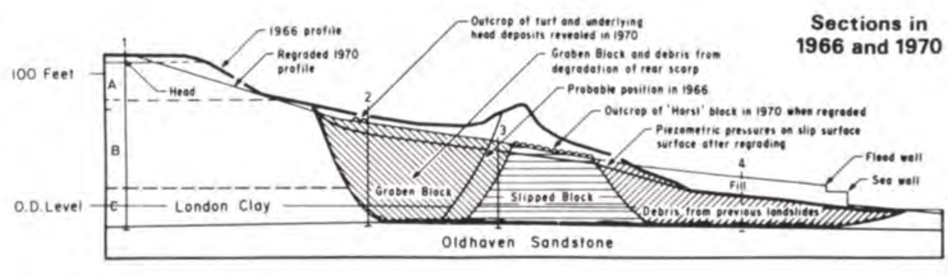


Fig. 4
Sections in 1966 and 1970

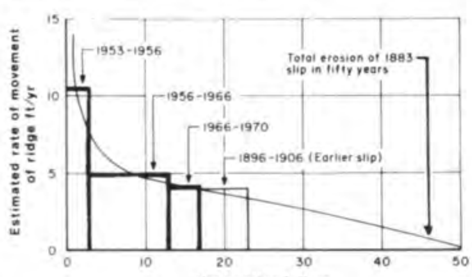


Fig. 5
Estimate of rate of movement of crest with time

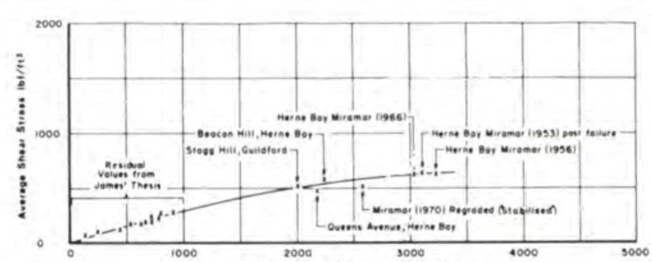


Fig. 6
Variation of mean shear stress on failure surfaces with the mean normal effective stress

to give residual and peak shear strength parameters:

(a) The situation immediately prior to failure in 1953 (both with and without the progressive failure approximation)

(b) The situation some weeks after failure

(c) The geometry existing in 1956

(d) The geometry existing in 1966

and the results of these analyses are presented in Table 1 below with the results of the stability analysis of the regraded profile included for comparison.

The residual strength analysis was carried out first and it was then established that the stresses in the 1883 slip remnants were appropriate to a ∅'r of 11° in 1953. Using this strength in an analysis (Fig. 1) gave peak parameters of C'=0, ∅'=20° in the intact clay (along the slip surface from B to D) but, following a suggestion by Hutchinson that stress relief may have allowed strains along the interface with the Oldhaven Beds, reducing the shear strength to the residual value (along B to C), a further computer run gave values of C'=450, ∅'=20 along the remainder of the failure surface, CD. This is thought to simulate the maximum influence of a possible progressive failure mechanism[1].

Possible errors

Inevitably in a survey such as this, where the conclusions are based on incomplete evidence, various alternative interpretations of the available data may be made and an assessment of the possible errors assists in checking the validity of the results obtained. These possibilities are:

(a) That unloading the failure surface may temporarily increase the residual strength[2]. The mean effective stress on the slip surface has been reduced, although the movements have probably been sufficient to restore the minimum value.

(b) Gross errors have been made in the evaluation of pore pressures. This must always remain a valid criticism where observations are lacking, but the model assumed from the recent records would seem to be most reasonable.

(c) Gross errors have been introduced by incorrect determination of the ground or slip-surface levels. In the case of the earliest profiles, the ground levels are probably accurate to within ±2 m, and this would appear to be sufficient for the analysis. The failure surface and most recent ground surface levels are known to a much greater accuracy and alternative toe positions chosen in the analysis do not significantly affect the results. It is highly unlikely that the failure surfaces detected were not those on which movement took place. However, movement could have been continued by a compound movement where the remnants of the 1883 slip moved first, precipitating further movement of the remainder. If this were the case, the true residual angle of shearing resistance would be higher than that calculated from the stability of the whole mass.

(d) The neglect of end effects in the stability calculations is thought to be reasonable as the slip is very wide and the geometry is sensibly constant throughout this width.

Fig. 7
View of 1953 slip from the centre looking east
Photo: Courtesy Soil Mechanics Ltd.

Fig. 8
Failure surfaces of 1953 slip

(e) Neglecting the work done in internal deformation in the calculation probably gives rise to serious errors (particularly in respect of the first failure, where a progressive failure mechanism was probably in action). However, until a method of analysis based on the stress/strain/volume change behaviour of the soil is available, this inaccuracy must be accepted.

Additional information

In 1970, regrading effectively destroyed the Miramar landslide as the subject of further study, but during this process considerable evidence was brought to light which assisted in the interpretation of the earlier surface observations and borings.

Conclusions

The study of the stability of the Miramar landslide, carried out as a part of the overall study of the cliffs at Herne Bay, has yielded a number of important pieces of data of the strength behaviour of London Clay. These are:

(a) Peak strengths: Two alternative assessments of the influence of progressive failure have given realistic upper and lower bounds for the peak shear strength parameters of London Clay in the field at high stress levels where there were no previous data.

(b) Residual strengths: A large number of analyses showed that in the field, as in Bishop's ring shear tests[2], the residual angle of shearing resistance is stress dependent and values have been established at hitherto unprecedented stress levels by means of a very well documented and instrumented case history.

Table 1: Results of stability analyses

Profile date	Average shear stress lbf/ft²	Average normal effective stress lbf/ft²	Average r_u value	Inclination of interslice forces ° **	Comments ***
1953*	3050	1020	.33	15 to 18°	See note 1
1953	680	3106	.40	9.76°	∅'m=12.3° pore pressures assumed
1956	615	3223	.38	9.40°	∅'m=10.8°
1966	618	3030	.37	9.63°	∅'m=11.5°
1970	528	2563	.43	7.40°	∅'m=11.6°

* Two alternative toe positions have been taken: (a) passes along the old slip surface of the 1883 slip, and (b) passes through the landslide debris. The most critical was found to be (a). ∅'r along old slip surface assumed to be 11°.

** Taken as acting at a uniform inclination to the horizontal and evaluated in terms of total stresses.

*** Obviously, the mobilized angle of shearing resistance ∅'m is equal to the residual angle if the factor of safety is unity.

Note 1: With residual strengths along AB; C'peak=0 and τ=1675, σn'=4600 along BD, however, with residual strengths along AC; C' peak=450 with τ=1200, σn'=2060 along CD. In both cases the orthodox value of ∅' peak=20° has been assumed.

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Tubular steel microwave towers for Nigeria

Mike Shears

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Introduction

The microwave radio towers described in this article form part of a contract for the provision of a microwave radio relay system for Step III of the Nigerian National Telecommunications Development Plan. The main contract, one of the largest of its kind ever placed with a British company, was awarded to The General Electric Company Ltd. in 1968 and, in addition to the radio equipment, comprises the supply and installation of aerial supporting structures, aerial systems, power supply equipment, equipment buildings and all associated civil works.

The Step III microwave and VHF radio relay system utilizes 93 entirely new stations, as well as the refurbishment of existing facilities and the addition of updated aerial systems at some link stations, to provide a network covering nearly 4000 route kilometres. The new system will be integrated with existing transmission systems and telephone exchanges in Nigeria, many of which, including the 760 km. Step I network, were also provided by The General Electric Company Ltd. When completed towards the end of 1972, the network will connect all the large centres of population in Nigeria, as well as many smaller communities, to give the entire country an advanced telephone service. The nationwide extent of the system is indicated by the map in Fig. 1, which also gives the transmission frequencies adopted on the various routes of the network.

The Step III contract is a 'turnkey' project in which The GEC is responsible for all aspects from the initial route surveys, and the systems planning operations, to bringing the network into service. The pre-contract estimating was based on a preliminary aerial survey over 565 km. of the network, the remainder being covered by map surveys, but the final station locations and optimum tower heights could not be determined until the results of a detailed ground survey had been processed. The ground survey teams were able to begin field operations in the autumn of 1968 in all regions except the Eastern States, where access to the sites was not possible until early 1970. Due to the requirements of the main contract programme, however, the tower designs for fabrication tenders had to be based on provisional heights.

The final tower heights and orientations became available, route by route, at intervals, as information was received from the survey teams during the first few months of the currency of the fabrication programme. Design details were then issued to the fabricator in batches according to a predetermined sequence. The tower design philosophy adopted to deal with these circumstances is described later in this article.

Tower design specification

The main contract specification called for self-supporting towers at the terminal stations on the network, but allowed consideration of guyed masts for the intermediate, or repeater stations. In view of the wide range of tower heights and the variety of aerial arrangements proposed, however, and coupled with the limited site area available at a number of the stations, it was finally decided to provide self-supporting towers throughout.

The specification also required that the supporting structures be designed with sufficient strength and rigidity to permit system operation without noticeable transmission degradation under the action of wind velocities of 38 m/s for structures located north of the Niger and Benue Rivers and 33.5 m/s for structures located south of these rivers. In each case the design wind gust velocities were defined at 10 m above ground level, and were assumed to vary with height according to a power law profile.

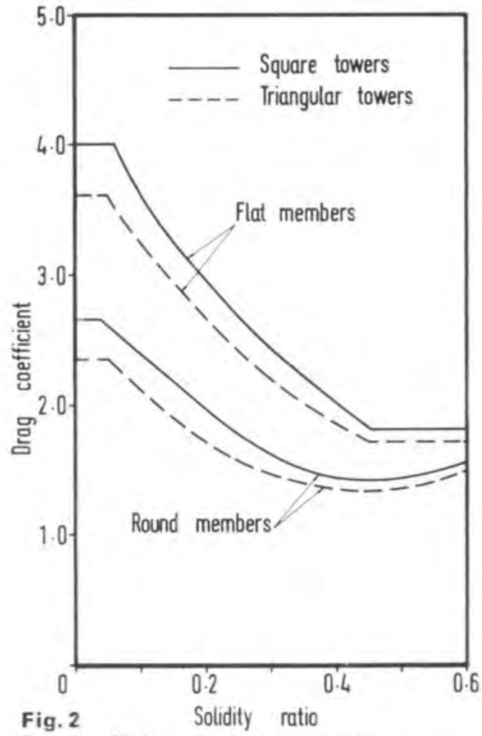


Fig. 2
Drag coefficients for latticed towers

For the consideration of tower rigidity, the operational design requirements were expressed in terms of displacement of the aerial mounting point in azimuth and elevation, taking account of the relative displacements of the aerial with respect to the supporting structure, such that the resulting radio beam displacements would not exceed the half-power points at the specified design wind velocities.

Two types of microwave equipment are used in the system with transmission frequencies of 6 GHz and 2 GHz, both utilizing 3.65 m diameter GEC parabolic dish aerials, in addition to Yagi type aerials for VHF transmissions. The specified radiated beam displacement requirements were as follows:

Aerial frequency	*Total radio beam*	*Tower*
6 GHz	±0.55°	0.45°
2 GHz	±1.30°	1.20°
VHF	±10°	10°

Wind loading

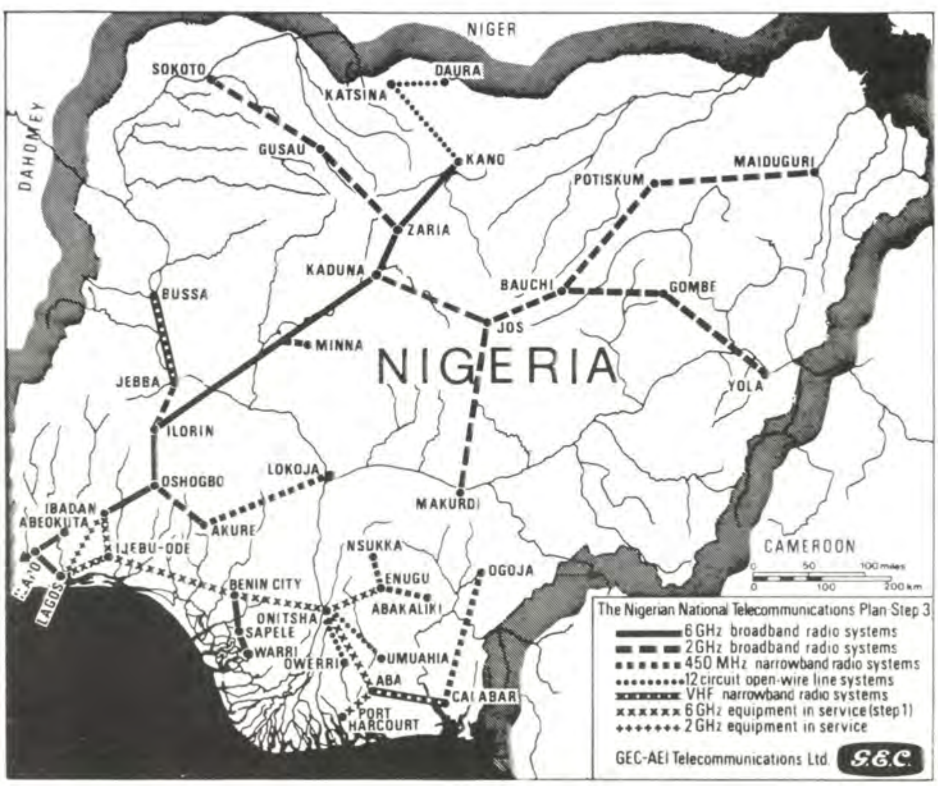
The resulting wind forces acting on the tower body structure were obtained using composite drag coefficients consistent with the plan shape and the type and arrangement of members used in the construction. The drag coefficients used in the design exercises were expressed in terms of solidity ratio; the ratio of area of exposed steelwork to envelope area of that portion of windward tower face under consideration (Fig. 2). The effect of angle of incidence of the wind to a tower face was allowed for by means of amplification factors, again expressed in terms of solidity ratio, applied to the calculated drag coefficients.

The wind forces produced by the aerial arrays were determined from wind tunnel measurements supplied by the manufacturers, and the forces acting on individual structural members, ladders and feeder cables, etc., were assessed using published values of lift and drag coefficients for isolated members.

Choice of tubular towers

The decision to use tubular steel members for the construction of the towers was reached at the pre-tender stage of the main contract.

Fig. 1
Nigeria Step III route map



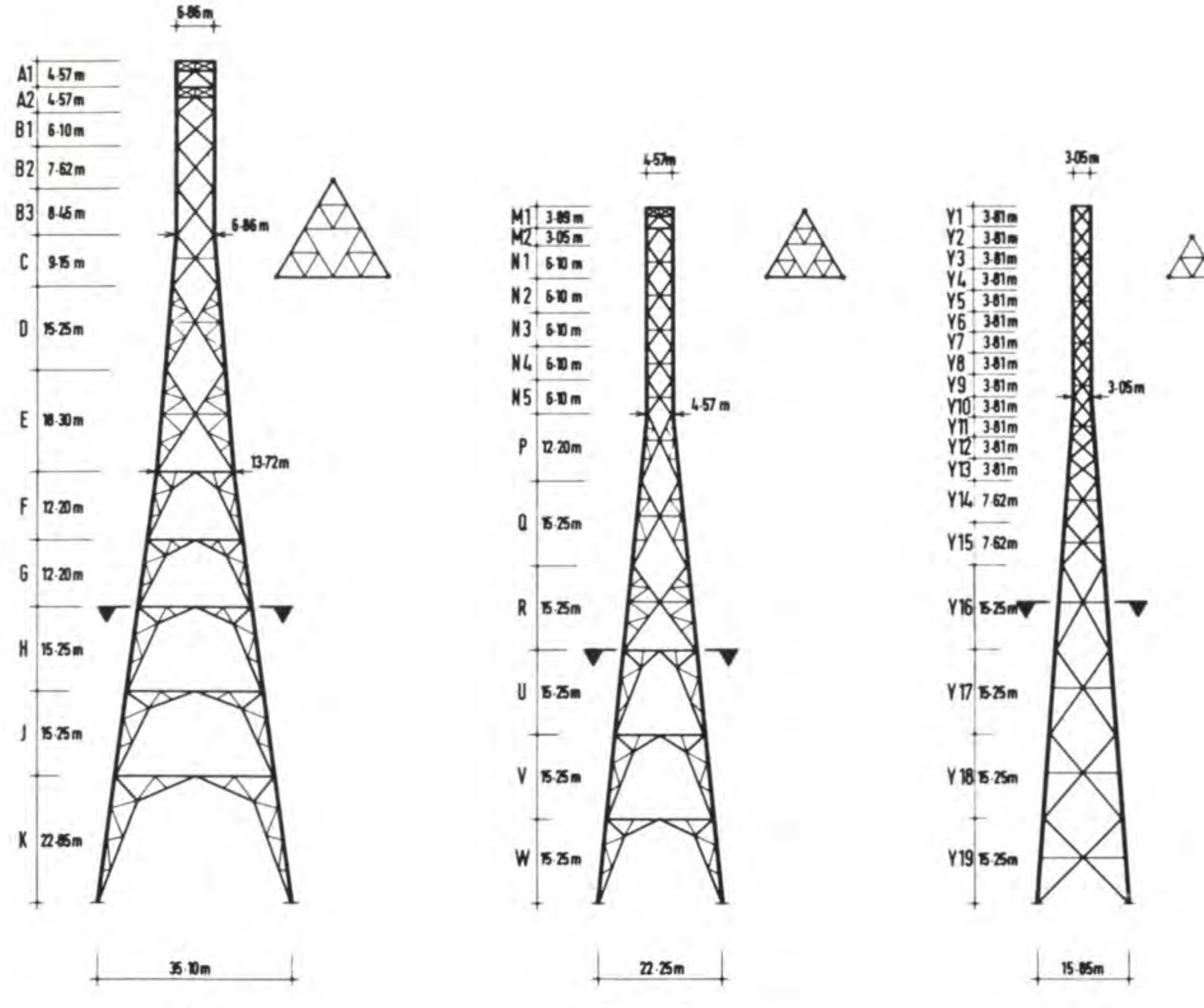


Fig. 3
Basic tower profiles for Heavy, Medium and Light route families

It had been realized that there were certain advantages to be gained from the use of tubular construction, particularly for towers of triangular plan shape, both in terms of economy and appearance.

The wind forces attracted by open, latticed towers composed of round tubes are measurably less than for towers of similar geometry composed of flat, open members. Furthermore, there is an even greater reduction in wind force for triangular shaped towers compared with the more traditional square towers (Fig. 2). Combining these effects, it is found that the wind forces acting on a triangular tubular tower can be as much as 50% less than for an equivalent square tower constructed of angle members.

The wind forces attracted by the aerials, however, are independent of the type of supporting structure, and the proportion of total wind force carried by individual tower members is somewhat greater for triangular towers than for square towers.

The final comparison also depends on the type of aerial to be supported, since there is a marked difference in rigidity between the two types of tower. On balance, however, the final design forces in corresponding members for the two tower types are similar in magnitude. Taking into account the superior compression load-carrying properties of tubes together with the fact that there are proportionately fewer members in the case of triangular towers, it is found that the weight of these towers is substantially less than for square ones of similar dimensions.

As a confirmatory exercise, parallel designs were developed, during the pre-tender stage of the main contract, for sets of towers based on the two types discussed. Tenders were obtained for both fabrication and erection, and comparative shipping costs established. The result indicated a clear economic advantage in the use of tubular towers.

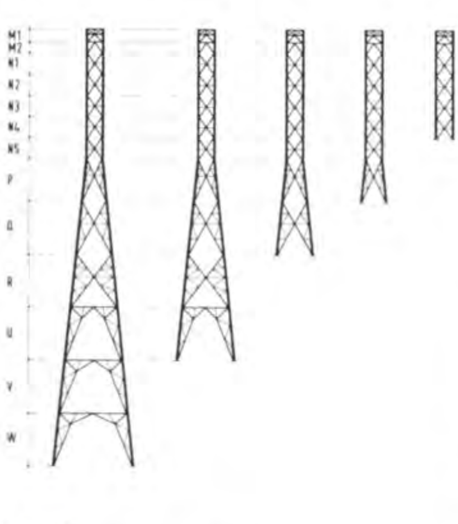


Fig. 4
Typical range of Medium route towers

Development of standardized tower designs

The need to obtain tenders for the fabrication and supply of tower and ancillary steelwork before final details of the structures could be made available, meant that initial designs had to be based on provisional information obtained from the preliminary aerial and map surveys.

Due to the highly competitive nature of international telecommunications projects, the tower designs had to meet the contract specification in the most economical manner, preferably on a station by station basis. At the same time, however, the designs were to be sufficiently flexible to accommodate the final tower requirements with the least possible alteration of details and without delaying the fabrication programme.

Solutions to satisfy these conflicting requirements would have been quite simple in isolation. For example, the towers could have been designed individually as final details became available from the survey teams, thereby providing minimum weight structures within the framework of a set of consistent tower profiles to minimize fabrication costs. This procedure was not possible, however, without seriously delaying the fabrication programme. Alternatively, a complete set of final tower designs could have been produced prior to receiving the final station details by basing all designs on the tallest possible tower having the most severe aerial loading. Such a solution would allow fabrication to proceed unhindered, but would not have been economic.

The compromise solution finally adopted was to group the towers into three distinct families according to the primary aerial system to be supported:

29 Heavy route towers – 6 GHz microwave system

48 Medium route towers – 2 GHz microwave system

16 Light route towers – VHF system.

The need to separate the two microwave systems arose from the radiated beam displacements specified for the aerials, and not from any difference in wind loading. The VHF Yagi-type aerial arrays, however, offer relatively little resistance to wind and are not subject to stringent rigidity requirements. These latter aerial systems are frequently carried by light guyed masts and clearly indicated a different class of support structure compared with the two microwave tower families.

Since the distribution of aerial heights and arrangements was fairly well established, it was possible to develop basic tower designs of standardized profile and geometry for each family. The final overall heights of the towers ranged up to about 143 m for the Heavy route, 123 m for the Medium route and 112 m for the Light route. Design and fabrication were carried out in imperial units.

Each of the three basic towers is composed of a set of standard tower panels (Fig. 3). The leg members and majority of bracing members are Circular Hollow Sections (CHS), although for economic or functional reasons some standard panels are hybridized by the part introduction of rolled steel angle bracings. The CHS sizes utilized by the three families of towers range from 42.4 mm diameter for the smallest secondary bracings to 323.9 mm diameter for the largest leg members. All CHS used are mild steel to *BS4360* Grade 43C.

Within any standard tower panel the overall member sizes and lengths are fixed, and gusset plates are identically shaped and drilled to receive any of the standard end connectors occurring within that panel. The geometry of each standard tower panel, therefore, remains identical throughout the range of tower heights, thereby enabling fabrication drawings, templating and jig production to proceed ahead of the release of final working details – one set of standard panels being sufficient for all towers in the family.

Fig. 4 shows a typical range of towers from the Medium route family.

Member loading variations within a given standard panel, due to differences in design, windspeed or the number and arrangement of aerials, are catered for as far as possible by making use of the further advantage of tubular construction, namely the range of different wall thicknesses associated with each tube diameter.

Extreme variations of loading beyond the capacity range of the standard member size, resulting from severe aerial configurations, are accommodated by a more appropriate arrangement of standard panels. For example, the upper portions of each tower type are parallel sided and contain a number of interchangeable panels to provide maximum overall height flexibility. Fig. 5 illustrates different arrangements of standard panels resulting in towers of identical height but having quite different load-carrying capabilities. For the example shown, Tower 1 has greater strength and rigidity than Tower 2.

Tower steelwork connections

All tubular bracing members are square cut and have T-pieces welded at each end for direct bolting to gusset plate connections. Seven standard T-connectors were developed to suit the various member sizes and load conditions experienced by the three tower families. Primary bracings rarely require more than two or three types of T-connector at any given panel joint, and all secondary and tertiary bracings are fitted with a single, identical connector throughout. A typical gusset plate connection is illustrated in Fig. 6.

Wherever rolled steel angle bracings are incorporated in the design, their sizes and connection details remain the same for all towers.

A number of alternative details was considered for the tower leg splices. The arrangement finally adopted consists of circular flange plates welded over the ends of the tubular leg, the connection being provided by a ring of high strength bolts (Fig. 7). To ensure full bearing between flange plates for the transmission of compressive loads, the plates were machined flat after welding. The splice positions within a tower panel, and the bolt pitch circle diameters, remain constant for all towers, although the number of bolts vary within a limited range according to the loading conditions.

A simple holding-down bolt arrangement is used to connect the tower legs to independent reinforced concrete foundations. To avoid the problems of non-standard detailing and templating in the base panel of each tower, the lowest leg splice in each panel was raised sufficiently to receive a standard base connector for attachment to the holding-down bolts (Fig. 8).

The use of standard details in this manner allowed all the tower body steelwork to be described by a set of tower panel drawings, with schedules listing the appropriate tubular wall thicknesses, end connections or splice types for each tower.

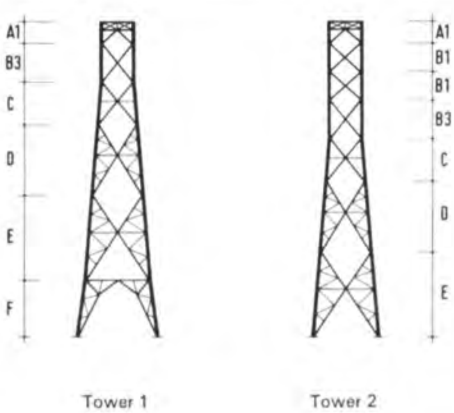


Fig. 5
Alternative arrangements of standard panels

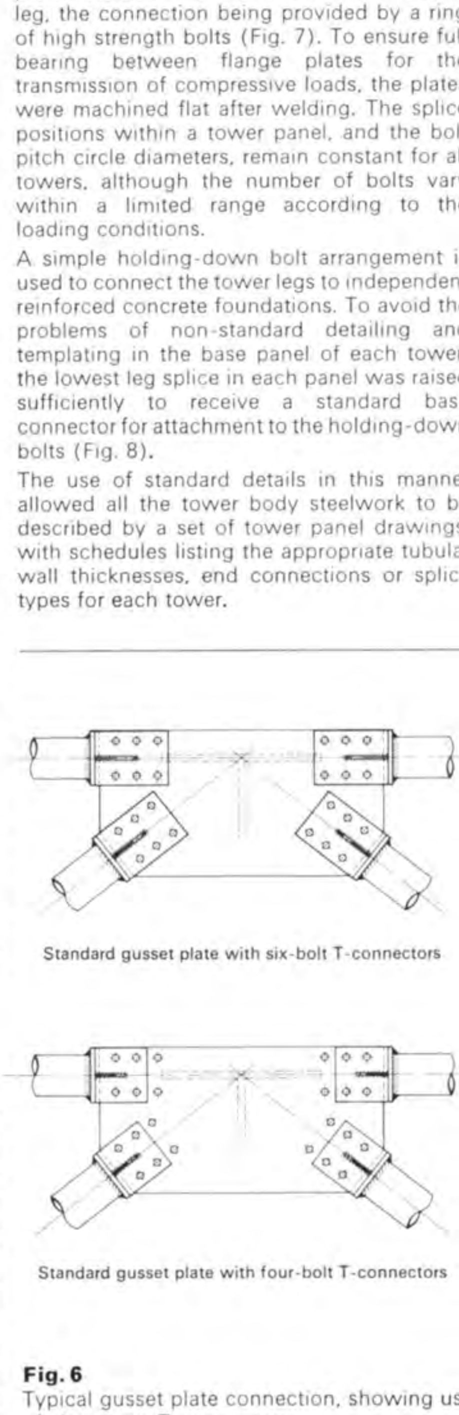


Fig. 6
Typical gusset plate connection, showing use of alternative T-connectors

Ancillary steelwork

The provision of ladders, platforms and aerial mounting steelwork depends on the site orientation of the tower and the distribution of aerial heights, both of which vary from station to station.

In order, therefore, to retain the standardized tower fabrication details, and at the same time provide a flexible arrangement of aerial mounting positions, all ancillary steelwork was designed to be clamped on to the tower body steel. In fact, the final locations of ancillary items were generally not available until after tower fabrication had commenced, so that the clamped-on principle allowed tower body fabrication to proceed unhindered.

Exceptions to the clamping arrangement included the platforms and aerial mountings at the top of each tower, where special aerial mounting panels were provided. Also, the ladder connections to the main plan bracings were made permanent by introducing replacement ladder support members in lieu of the standard bracings.

The clamping details were developed with the fabrication subcontractor, who produced prototype connections of each type for trial assembly and load testing.

Tower production, supply and erection

Following the receipt of tower and aerial requirements from data obtained by the field survey teams, the final designs and working details were issued in batches to the fabrication subcontractor, who then placed final steel orders, updated his own drawings and schedules, and produced any new jigs or templates.

Before commencing full fabrication production of any tower family, a master tower containing a complete set of standard panels and all types of ancillary steelwork, was trial assembled at the fabricator's works in Hereford (Fig. 9).

Due to the programme requirements it was not possible to check assemble the setting-out templates for the foundations against the corresponding tower base panels. These templates were fully triangulated, however, and all connections were made with close tolerance bolts. Each template was assembled before issue and checked by tape measurement, the accuracy achieved by the fabricator being better than 0.02% in overall length.

Tower and ancillary steelwork followed the delivery of foundation holding-down steelwork and setting-out templates, according to a sequence to suit the tower erection programme.

All steelwork was fully galvanized and treated to prevent white rust formation. Tubular members were galvanized inside and outside after fabrication by leaving vent holes in end flange plates or T-connectors, the holes being sealed later with special galvanized plugs.

In all, over 3000 tonnes of steelwork were supplied free on board for the new towers under the fabrication subcontract.

Shipping of all materials was arranged by The General Electric Company Ltd., who also maintained a depot in Nigeria, where tower steelwork was received, sorted and stored before being handed over to the erection subcontractor for delivery to site. A resident engineer at the depot dealt with damaged or missing steelwork, either by local repair and refabrication or by re-order from the UK. The use of standardized components for the towers allowed a degree of cannibalization of parts from one tower to another to take place.

Tower erection (Fig. 10) was carried out simultaneously at a number of sites on the network, all access roads, buildings and tower foundations having previously been installed by a separate civil works subcontractor. The speed of erection of the tubular towers (Fig. 11) depended on the complexity of ancillary steelwork, but was generally faster

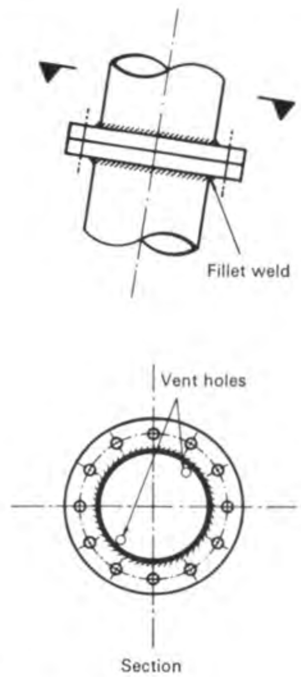


Fig. 7
Tower leg splice detail

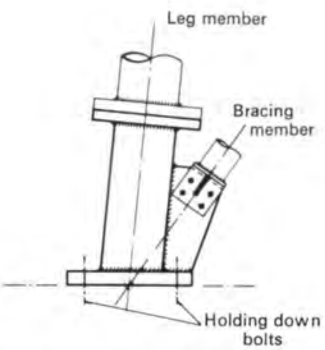


Fig. 8
Tower leg base-connector detail

than anticipated, the on-site erection time being sometimes less than six days for a 91.5 m tower.

Where required, the towers were painted and fitted with aircraft warning lights; otherwise the towers were left in their galvanized condition. Aerial and feeder cable erection was carried out by separate teams following behind the tower erection gangs. As routes were completed the aerials were panned and adjusted for final orientation before radio testing (Figs. 12 and 13).

The microwave towers described in this article demonstrate that the use of tubular construction can result in an attractive and economic solution in a situation traditionally dominated by other forms of construction.

The permission of GEC Telecommunications Ltd. to publish this article is gratefully acknowledged.

Credits

Main contractor to the Federal Government of Nigeria: The General Electric Company Ltd.

Architects for site building works: Godwin and Hopwood, Lagos.

Quantity surveyors for civil works: Messrs. Tillyards.

Structural and civil engineers: Ove Arup and Partners.

Civil works subcontractor: D'Alberto and Bogialla Ltd., Nigeria.

Steelwork fabrication and supply subcontractor: Painter Bros. Ltd., Hereford.

Tower erection subcontractor: Powerlines Ltd., Nigeria.

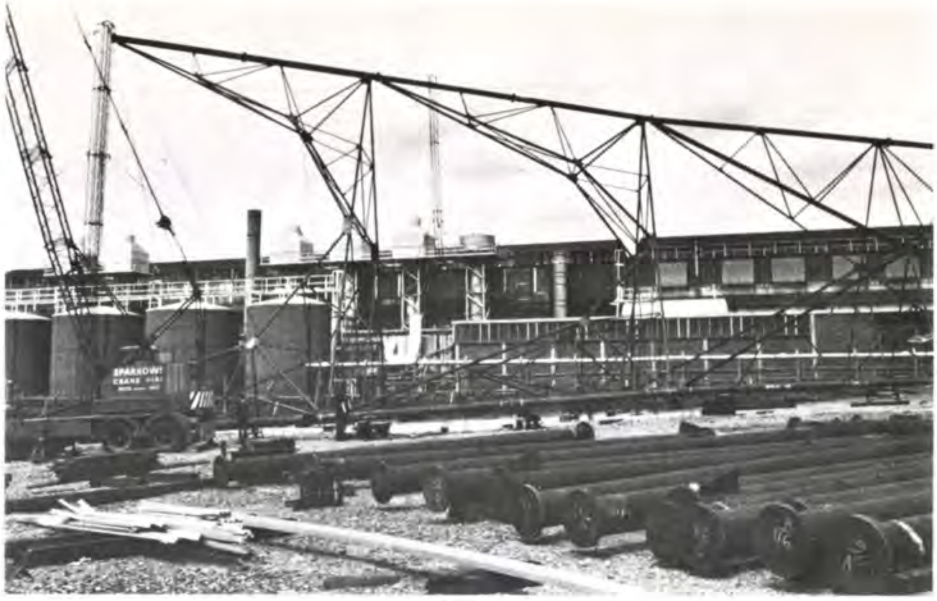


Fig. 9
Trial assembly of Heavy route tower

Fig. 10
Tower erection in Nigeria

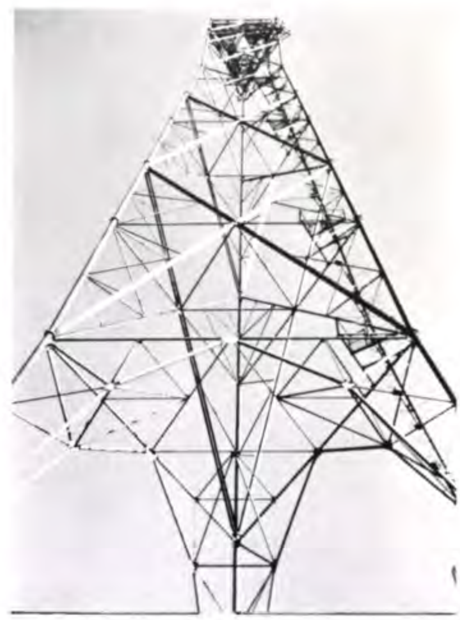


Fig. 11
Heavy route tower prior to aerial erection

Fig. 12
Completed Heavy route tower

Fig. 13
Top of Heavy route tower showing aerial mounting panel

Planning to avoid cooling tower misapplication problems

Des Gurney

This paper was given at the Cooling Water Association Symposium on 28 April 1972.

Introduction

Quoting from an earlier text[1]: 'It is unfortunate that the correct selection and application of cooling towers . . . is too frequently given scant atttention . . . and so often the cooling tower becomes 'nobody's baby'' – yet this is the ultimate instrument of heat sink for the cooling system – a failure here places the whole plant or process in jeopardy.

As engineers, we are expected to play all manner of tricks in respect of tower application and location; it is in the hope of providing useful guidance, based on some warning examples, that this paper has been prepared.

Performance expectation

Let us study first the essential expectations and fond hopes of the client and user – what does he want for his money?

For a given thermal duty, a cooling tower is expected to provide

1 Maximum thermal performance for lowest possible capital outlay
2 Long, trouble-free life at minimum cost for maintenance
3 Least possible disturbance to the environment and to other activities in the vicinity of the tower
4 Very often, a tower which looks good, and stays that way throughout its life; perhaps even an invisible tower!

The client and user, be he architect or industrialist, rarely realizes at the planning stage that he wants these things – but he soon remembers when he gets his tower!

We are all very well aware that these are conflicting requirements, and their interpretation, their achievement by careful evaluation and compromise, is often left in the wrong hands.

Responsibilities

Perhaps we should examine the responsibilities of the various involvements.

It is the responsibility of the designers of the site, and of the plant which it serves, to ensure that the cooling tower installed is suitable for its job in terms of its performance, application, location and appearance.

The prime responsibility of the manufacturer is to provide an efficient and dependable product at a viable price, guaranteed to meet the performance which has been specified, and to achieve this by good engineering compromise so that a sensible balance is obtained between the conflicting requirements, of which he is well aware.

While there are, unfortunately, very few cooling tower engineers, the majority of cooling tower suppliers (and there is a wide choice – perhaps three dozen in the United Kingdom) employ sales and applications staff who are well versed in the problems and who can, when given the opportunity, provide sound advice on the application and location of their product. But the salesman can only assess the information which he is given, and this rarely includes layout and planning material or sources of environmental pollution. At the same time, he must come to terms with the commercial pressures imposed by a competitive industry, and the less technical information is provided, the more commercial pressures will prevail.

Location and planning

Cooling towers are expected to be capable of being positioned almost anywhere:

on the roof
beside the boiler flues
behind screen walls
in the car park
in a courtyard
completely boxed in
in the basement
in a pit
underground.

This is all fine from the planning and architectural view – almost all things can be achieved provided that the engineer has control over time and money, *and* the details of planning which affect the proper performance of the tower. Of course he rarely has this control!

What is often forgotten is to compare alternative arrangements and types of tower in order to solve the problem.

Types of cooling tower

The various types of tower have been classified and their pros and cons outlined in other texts.[1,2]

We most often encounter towers of the mechanical draft type in the following configurations: induced draft counterflow, forced draft counterflow, single entry crossflow, double entry crossflow. Nowadays, we are not generally concerned with atmospheric natural draft towers unless they be very large, and of the hyperbolic type; since a previous author has examined these in detail in a recent paper[5], it is not proposed to repeat similar material here.

Each tower arrangement has its potential problems and vicissitudes, and there are some variations on the standard themes which can be useful in certain situations. In order to avoid more than a fair share of problems in a given installation, it is necessary to examine the location from two main points of view: the effect of the location on the cooling tower operation and the effect of the cooling tower on its immediate surroundings.

Effects of location on tower operation

It is necessary to evaluate the likely effects on the following parameters essential to cooling tower thermal performance:

(a) Free access for entering air
(b) Danger of recirculation of outlet air to inlet
(c) Uplift in air wet bulb temperature
(d) Air (and hence water) contamination
(e) Spoiling of fan performance
(f) Access for erection and maintenance due to the positioning of the tower in relation to its surroundings.

(a) *Free access for entering air*

Buildings and large items of plant, close proximity of other cooling towers and of solid enclosing walls, can all obstruct entry of air to towers.

Solutions include calculation of the free air access area required. For the plant designer to do this, which is his responsibility, tower manufacturers must declare the air rate requirements of their towers, together with air entering and leaving velocities. As a minimum, free area for air flow in any direction must be not less than equal to the gross air entry area to the packing. Multiple towers cannot share this free area – there must be sufficient area for the total airflow at a mean velocity of perhaps 60-90 m per minute.

Consideration of the various types of tower available can be of considerable benefit, along the guide lines given in Table 1:

Table 1: Various tower arrangements according to free air access

Number of sides with unobstructed air entry	Alternative tower arrangements
Four	Induced draft counterflow – Fig. 1
Three	Induced draft counterflow with increased height air entry – Fig. 1
Two Adjacent	See Fig. 2: Independent pair of forced draft towers; Specially designed induced draft counterflow
Two Opposite	See Fig. 3: Double entry induced draft crossflow; Single entry forced crossflow; Back to back pair of independent forced draft counterflow; Induced draft towers with increased air entry height
One only	See Fig. 4: Forced draft counterflow; Half of induced draft counterflow
None: a pit	Provided space permits, a sunken induced draft counterflow tower with horizontal air inlet baffles at deck level, and discharge air bonnets or cones (Fig. 5). Sunken crossflow or forced draft liable to excessive recirculation

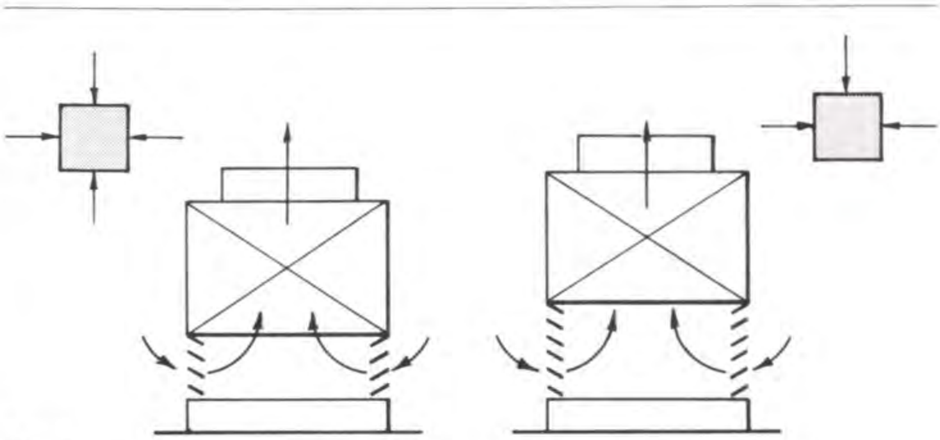


Fig. 1 Standard induced draft counterflow and Induced draft counterflow increased air entry

(b) *Recirculation dangers*

Buildings, large items of plant, other cooling towers and enclosing walls can all modify the mass movement of air in such a way as to cause recirculation; indeed, the bulk of an individual large tower, or bank of towers, can have this effect on itself, and so it is clear that the direction of the prevailing wind must be taken into account. Note that the 'prevailing wind' for the tower may not be the same as that prevailing for the site or area.

Another text[6] suggests that a tower's exposure to wind can be taken as open if it is located at a horizontal distance of at least 10 times the height of surrounding solid structures. Should a tall building or plant item be upwind of the tower, the air velocity will decrease as it passes the tower with a consequent tendency to entrain the warm moist discharge back into the downwind air inlet (Fig. 6).

Particular instances and partial solutions to the problems include:

(i) Towers downstream of large buildings when $L < 10H$: use high velocity discharge through cones.

(ii) Banks or batteries of towers behind screen walls: mount batteries parallel to prevailing wind, and never in series down wind (Fig. 7). Provide fully louvred enclosing walls with ample free area for entering air. Use only high velocity discharge, preferably with cones or bonnets. In some instances, a cure may be achieved by fitting decks or baffles at the top of the towers between enclosing walls and towers, also between cells; this should be considered palliative and not as a design solution (Fig. 8).

(iii) Single entry crossflow towers should be installed sideways – on to the prevailing wind (or modified wind direction due to surrounding structures) during critical performance periods. Large baffles fitted to top of tower can ameliorate recirculation, but again, this is not a design feature, merely a solution to a problem (Fig. 9). Single entry crossflow towers are not recommended for use on critical design duties because of the orientation problems.

(iv) Induced draft crossflow towers have high air velocities at the upper sections of inlet air louvres, and a pressure pattern is created which can encourage recirculation (Fig. 10). For this reason, extended discharge cones are normally fitted to these towers, and are frequently designed as pressure recovery stacks thus aiding fan performance.

(v) Buried, or partially buried, counterflow induced draft towers exhibit similar effects to the induced draft crossflow tower. Again, extended discharge cones should be fitted, together with decks or baffles, at the top of the tower and there should be ample air entry at the tower base.

(vi) Forced draft counterflow towers have lower air leaving velocities so that contributory factors increase the recirculation risks. Choice of location is again very important. Therefore the fitting of extended discharge bonnets and top decks may be of assistance (Fig. 11). Ample free air entry area is essential at low level. Forced draft towers with opposed fans operated on sequential thermostatic control are subject to recirculation unless fitted with flow reversal dampers.

Provided that the dangers of recirculation are made known to the tower supplier, he may be able to make allowances for the condition in his selection. This will normally result in a larger, more costly tower, or increased fan horse power, or a combination of both. Addition of discharge bonnets and higher fan stacks (cones) can sometimes reduce fan horse power due to the gain in 'stack effect' but these gains will be marginal.

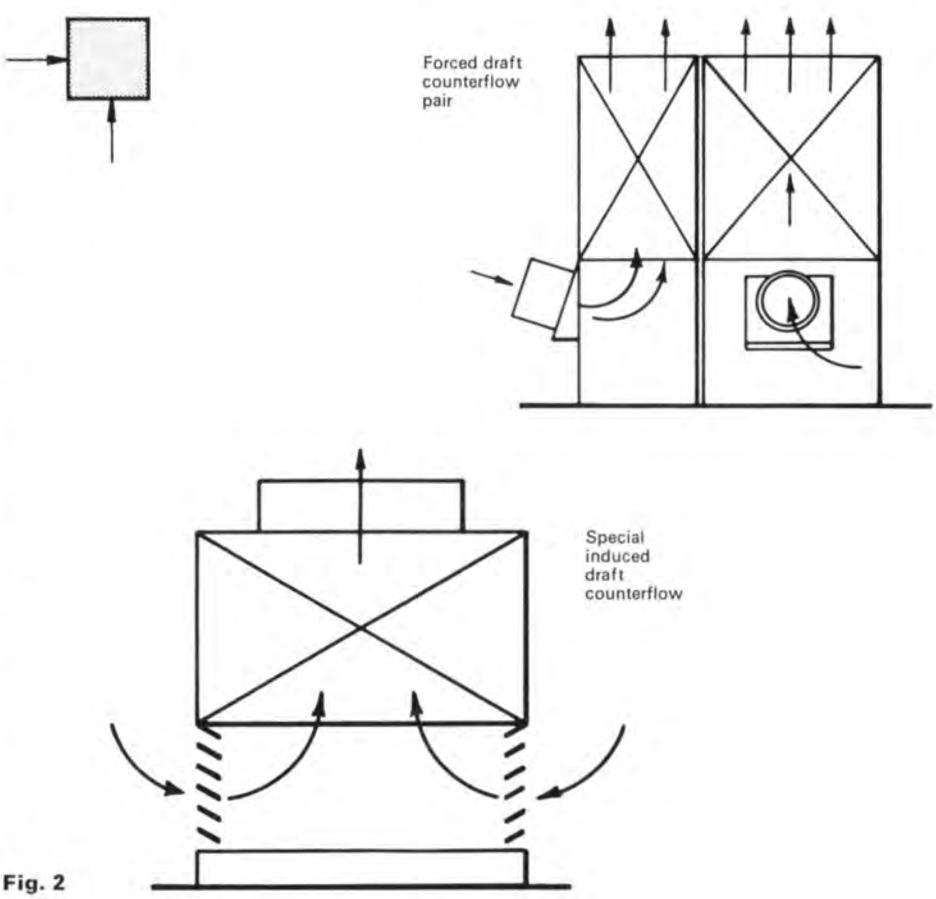


Fig. 2

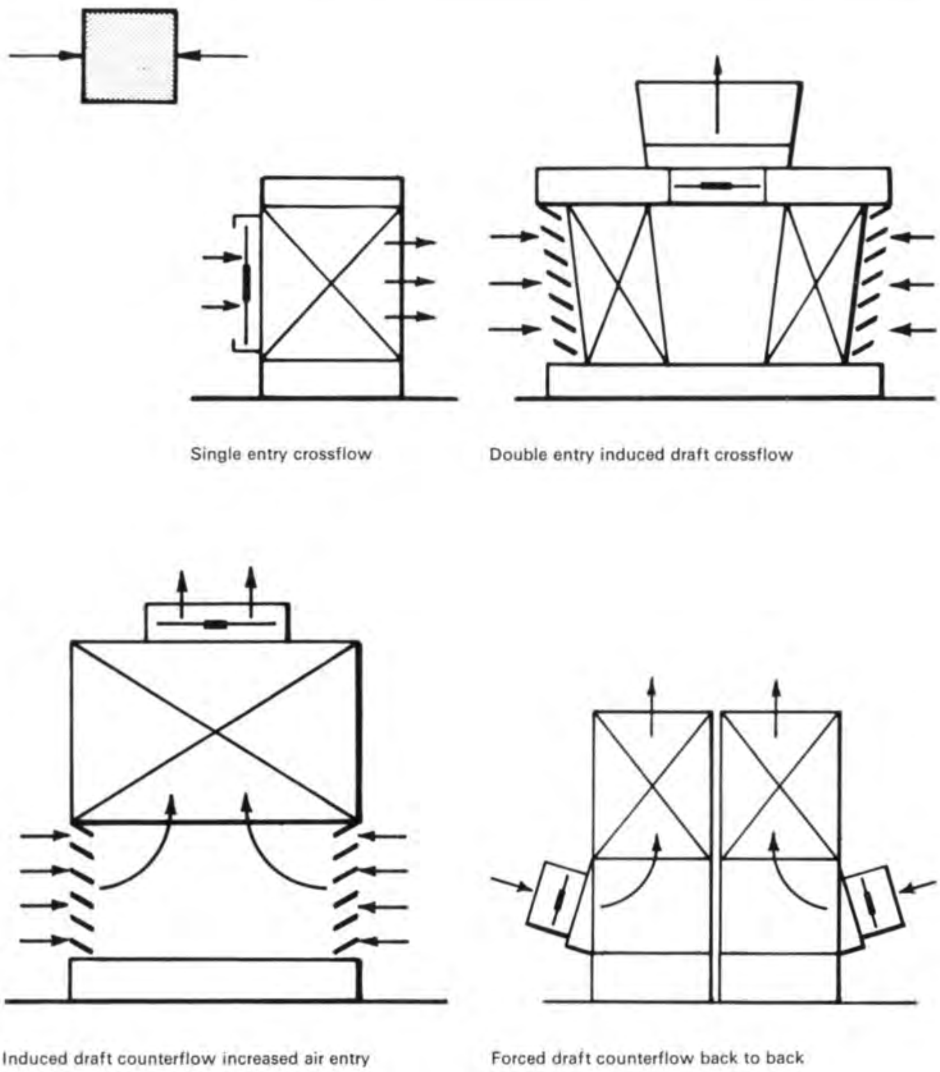


Fig. 3

It is suggested that any space and cost penalties, avoided by not taking precautionary measures against recirculation, will be more than offset by the increased costs and embarrassment caused by subsequent lack of thermal performance and remedial action which will have to be taken.

(c) *Uplift in air wet bulb temperature*
The dependence of the water cooling tower upon the wet bulb temperature of the entering air is well-known. What is not so often appreciated is the severe penalty on thermal performance which results from incorrect selection of this leading design parameter in terms of the effects of surrounding features, both natural and man-made. Other texts[1,6] should be studied for detailed information, since space permits only brief comment here. Situations in which the wet bulb will be increased include the following:

(i) Recirculation: Kunesch[5] in his recent paper quotes numerical examples which indicate the seriousness of this problem.

(ii) Air discharge grilles from building air conditioning, process steam discharges and effluxes from other cooling towers, from hotel bathrooms, laundries and so on, can adversely affect cooling tower thermal performance by elevating the entering wet bulb temperature.

(iii) Close presence of dense vegetation, particularly if the tower is surrounded by damp natural growths.

Certain advantageous situations can arise in which the wet bulb is reduced:

(i) Wet bulb reduces with altitude: the correction suggested for the British Isles [1,6] is about 0.5°C per 100 m of elevation.

(ii) The presence, upwind, of large stretches of water actually can have the effect of lowering the incident wet bulb.

If the cooling tower supplier is aware of the sources of possible influence on wet bulb temperature in the neighbourhood of the site, arrangements can be made to take these into account in the design and selection of the tower.

(d) *Contamination of air and water*
Tower performance can be adversely affected by entrainment of foreign matter of various types. It has to be remembered that the cooling tower is a highly efficient air washer:

(i) Entrainment of fumes from boiler flues will reduce the pH of the circulating water, resulting in rapid deterioration of the piping, heat exchangers and the tower itself, due to corrosion.

(ii) Other processes and effluxes, particularly oil and diesel fumes, will affect performance by coating the system with greasy deposits. Entrainment of cement dust from adjacent building operations is a common source of trouble.

(iii) Contamination by entrainment from biological sources such as kitchens and bakeries, laundries, paint spray booths and printing processes, will result in organic growths which will attack both the system and the tower itself. Failure to apply remedial action could result in total blockage and structural collapse of the tower internals.

(iv) Similar effects can result from blockage by leaves, catkins, bird feathers and droppings. The high velocity intakes to forced draft fans are particularly prone to picking up rubbish, leaves and sheets of plastic film.

(v) Contamination of the water can also arise from mixtures of metals in the cooling water circuit, resulting in products of corrosion and in metals dissolving into the water and thus accelerating corrosive action. In particular, zinc (galvanizing) and aluminium should not be used in circuits containing copper condenser tubes.

The general solutions and precautions to be taken include:

(i) A careful study of the proposed location in relation to both the plant or building which the tower is to serve, and also a survey of possible sources of trouble in the surrounding area.

(ii) Consideration of the use of air inlet filters (these must be regularly cleaned or water will be held up within the tower). In heavily air contaminated areas, such as steel works, the use of bypass water filters may be advisable.

(iii) In areas of heavy environmental pollution, consideration of the use of closed circuit cooling water circuits using either air blast (fin fan) coolers with subsidiary sprays under elevated ambient air temperatures, or alternatively closed circuit coils within a cooling tower concept, making the necessary detailed studies.

(iv) Planning for effective water treatment and regular blowdown, as necessary. The advice of a recognized water treatment specialist should be obtained. Mixtures of metals in construction should be carefully studied and incompatible mixtures avoided – water treatment is not a panacea in such cases.

(e) *Reduction in fan performance*
The effect of altitude on air pressure must be taken into account in the determination of fan performance.

Restriction of air inlets, dirty air inlet louvres or filters, dirty or maladjusted packing and water distribution, blocked or dirty eliminator sections, and restriction of air discharge, all adversely affect the cooling tower performance.

All the relevant factors previously discussed should be taken into account in order to reduce these effects – in particular the proposed location should be studied for secondary obstructions to air flow such as beams, cills, parapets, and items of equipment which may not be installed according to plan. In particular, structural members such as beams can create serious vibration problems by disturbing the flow of air over the leading edges of the fan blades.

(f) *Access for erection and maintenance*
The proposed location should be studied to ensure that the supplier can indeed erect the tower, and that the essential normal maintenance is possible, with a sufficiently large drain to carry both water and sludge, and a convenient hose connection to make washing down possible during draining. The functional design features of the water storage basin are of prime importance in the ability of the system to operate consistently and correctly.

Effect of tower on its surroundings
It is advisable to study the results of the following effects which may create problems within the immediate surroundings of the tower:
(a) Condensation of moist air discharge
(b) Downwind vapour drift
(c) Windage and carryover
(d) Noise from fans and water
(e) Weight of tower and water
(f) Results of accidental leakage
(g) Appearance in relation to surroundings.

In all cases, the best solution is to select a location which avoids these problems but, when this is not possible, other measures must be investigated.

(a) *Condensation of moist air discharge*
Since the discharge air is to all intents nearly 100% saturated with moisture, at a temperature above that of surrounding surfaces, condensation on nearby walls, windows and plant items is a near certainty. The results can vary from distasteful nuisance to actual damage, including:

(i) Staining of buildings, including salt deposition and biological growths
(ii) Corrosion of metal surfaces
(iii) Etching of glazing.

Use of lower water loadings (larger tower plan area and greater vapour dispersal) use of extended fan cones to carry the nuisance past the surfaces and also tilting the fans away from critical areas, have all proved effective.

(b) *Downwind vapour drift*
Even when condensation on critical surfaces is avoided, the warm moist air stream will condense, at periods of critical dewpoint, into a cloud of vapour which can persist for considerable distances downwind. In really difficult situations, such as near aircraft runways, gas burners have been installed within the fan discharge stack – but the cost of their use is very high. Fine nylon mesh screens have proved effective in one instance known to the author, by causing a percentage of the moisture to condense above the tower.

(c) *Windage and carryover*
At times of high wind, water may be blown out of the bottom of towers unless crosswind baffles are fitted. This is a frequent cause of icing of nearby paths and roads in winter, when the tower is situated too close to such routes. It is recommended that a distance of some 6-9 m should be allowed between large cooling towers and roads, according to the height of the air inlets, because the effectiveness of crosswind baffles varies.

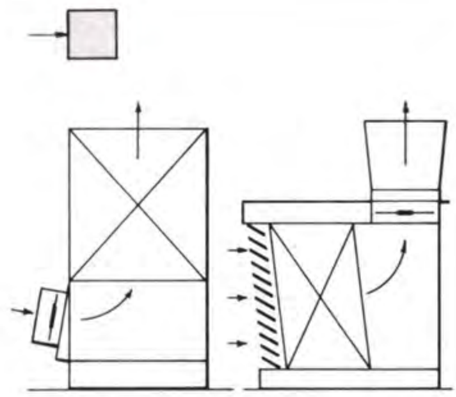


Fig. 4

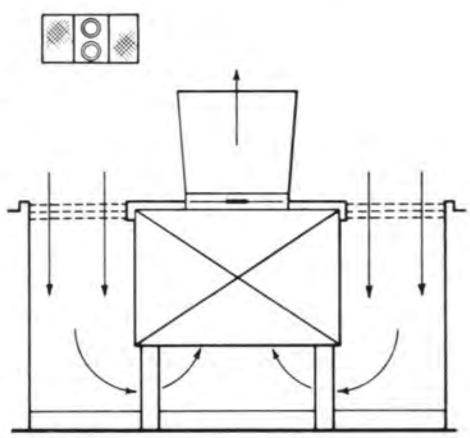


Fig. 5
Sunken induced draft counterfoil air inlets horizontal

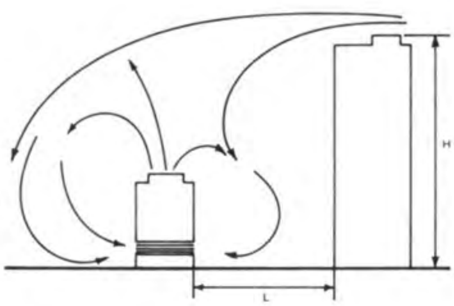


Fig. 6
L <10H increases recirculation

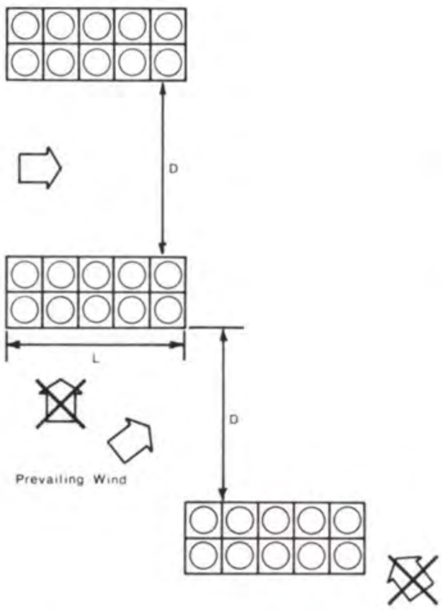


Fig. 7
Orientation of multiple tower banks, D to be < L

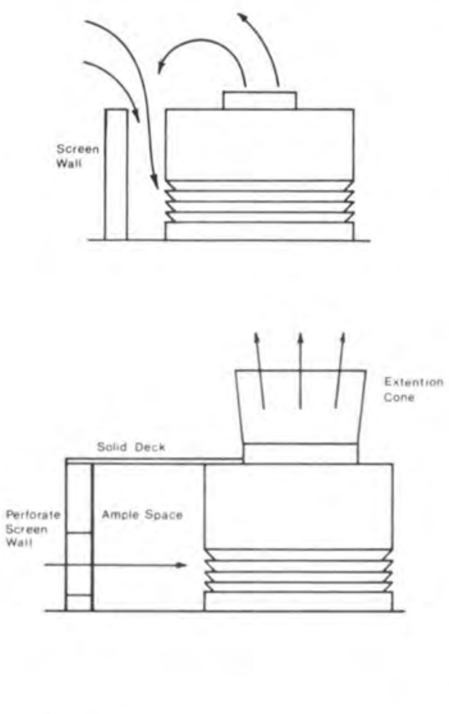


Fig. 8
Towers installed behind screen walls

Droplets of moisture can be tossed from the tops of high discharge velocity towers even when eliminators are fitted. The combination of concentrated impurities in the water and sunshine will mark the paint finishes on motorcars and plant. Unless the air discharge velocity is low, and very effective eliminators are fitted, towers should not be located close to car parks and roadways used by the general public.

(d) *Fan and water noise*
This subject and its problems and solutions, are dealt with in some detail in other texts[1,3]. It is sufficient here to emphasize the highly emotional response evoked by unacceptable noise levels and to make the following reminders:

(i) Running a fan continuously at part-speed at night is more acceptable than cycling it on and off.

(ii) Quiet cooling tower performance can be achieved using slow-speed or quiet running centrifugal fans and motors if the supplier knows of the problem at planning stage. Also, both inlet and outlet silencers are available as standard accessories on some makes of tower.

(iii) Fan cooled motors are more than ordinarily noisy and should not be used.

(iv) Tower water noise is real, but can be modified by use of anti-splash decks in the surface of the water basin. This should be at the operating water level which will be below the level of the surface when the tower is not in use.

(v) Towers should not be mounted in noise-sensitive courtyards since reflection increases the noise level by up to 15 db.

(vi) Fan/motor/gearbox noise can be transmitted through the structure of the plant and building, and the usual precautions taken for moving plant items apply also to cooling towers. Ask, what is situated below the cooling tower?

(e) *Weight of tower and water*
It is not always recognized that the operating weight of a cooling tower is high, that the quantity of water held within the packing and contained within the storage basin of even a packaged tower may well equal the weight of the tower itself. Suffice to say that the foundations or structure beneath the tower must be capable of carrying the weight of tower *plus water!*

(f) *Accidental leakage*
Once again, ask, what is underneath the tower?

The design of the piping circuit between the cooling tower and the plant which it serves should be studied to determine the quantity of water which will drain down into the tower basin when the pumps stop. Can the basin accept this water without waste through overflowing? Draining of the heat exchangers in the plant should not be permitted since other operating difficulties arise as a result.

(g) *Appearance*
The engineer, as well as the architect, should be concerned about the appearance of the tower, both when new, and also after some years of operation with minimum maintenance. Quite often the architect does not appreciate how large a cooling tower can be, nor that there is such a wide choice of types, arrangements and finishes. There is full scope at the planning stage, but very little by the time the engineer is finalizing the details.

There are many examples of towers totally unsuitable for their surroundings, which become an eyesore and a sad monument to the lack of care, understanding and interest of the building designers and site planners. Nobody's baby grows up with a very sore thumb!

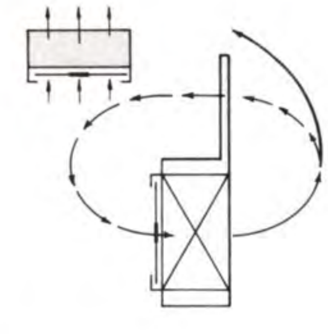


Fig. 9
Single entry crossflow

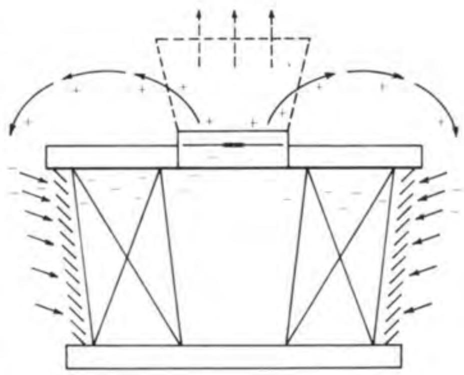


Fig. 10
Crossflow induced draft

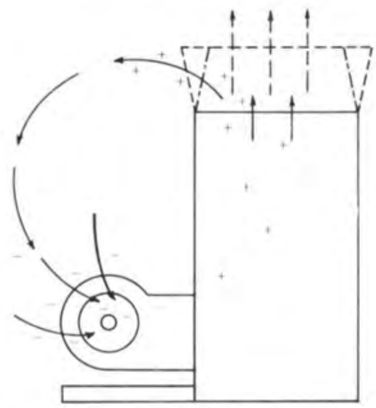


Fig. 11
Forced draft

Fire precautions

It is not always recognized that a cooling tower is a very real fire risk. There are many cases of destruction during the plant construction phase, from welding sparks or due to a casually dropped match or cigarette. A timber tower which has dried out during a period of maintenance is easily set alight – and the natural draft effect quickly fans the flames. In and on buildings, sprinkling or drenching of the outer casings may be advisable. In general, it is common sense not to allow a timber packing to dry out completely.

While there is still no standard for fire precautions in respect of cooling towers in the United Kingdom, the National Fire Protection Association in the USA publishes NFPA Standard No. 214 which is a useful basis for study of underlying principles.

Appendix 1 – Application checklist

1 Select as first choice the best possible position, checking for:

(a) *Effect on tower*

Free access for entering air, ample perforation
Recirculation danger (L< 10H), need for decks
Prevailing wind, orientation of tower batteries
Air contamination; other plant items, effluxes, filters

Uplift in air wet bulb

Spoiling of fan performance due to position of tower in its surroundings

The need for fire precautions

(b) *Effect of tower*

Condensation of moist air discharge; cones and bonnets

Downwind vapour nuisance

Windage and carryover nuisance

Noise from fans and water; silencers, part-speed motors, anti-splash decks, anti-vibration insulation

Weight of tower and water

Dangers from accidental leakage

Appearance and visual suitability.

2 Evaluate any problems and compare alternative locations.

3 Examine alternative types, arrangements and finishes of tower.

4 Provide maximum useful information to potential suppliers – see checklist for specification to suppliers.

Appendix 2 – Checklist for specification to suppliers

1 *Performance – specify:*

(i) The water flow rate

(ii) The inlet and outlet water temperatures

(iii) As a check, the heat to be removed

(iv) The design ambient wet bulb temperature

(v) Altitude of site

2 *Location – give:*

(i) The area or town in which the site is located

(ii) The location, i.e. whether at ground level, on or within a building.

(iii) A guide to surrounding environment, e.g. built-up areas, woodland, harbour, etc., which may affect performance or selection of materials. Adjacent railways, process effluxes, and industrial chimneys should be mentioned.

3 *Limitations*

Are there any restrictions in height, plan area, or access for erection which may affect the type of tower offered? Is there a preference for a particular type, e.g. induced draught (low plan area), forced draught (limited air access), cross draught (height limitation) should be stated.

Specify also:

Limitations due to noise (adjacent residential areas at each side or below)

Limitations on carryover due, for instance, to windows adjacent to discharge

Limitations in working weight due to structural considerations for a roof-mounted tower

Restricted access for air inlets, and dangers of recirculation.

4 *Construction*

The enquiry should state:

Whether the casing is to be of

(a) manufacturer's supply

(b) builder's work supply

Whether the basin is to be of

(a) manufacturer's supply

(b) builder's work supply

If (a) in either case, a statement of materials should be requested, or specified, in the enquiry, as should a description of ancillary fittings such as drains, purge connection, make-up valve, etc.

If (b), the enquiry should specify cover for supply of all necessary drawings to enable the contractor to construct the casing and/or basin, including the necessary baffles, filters and drains to trap and remove sludge.

Preference for any material, e.g. type of packing, should be stated. Restrictions in materials (study mixtures of metals due to other equipment).

When the tower is to be installed in the open, the enquiry should state whether ancillary items such as immersion heaters, ball valves, strainers, fan guards, cover, thermostats and motor starters are to be included in the quotation.

5 *Services*

The enquiry should state the volts, phase, cycles of the electricity supply, and should, if possible, give the source of the make-up water if this is other than town mains supply, e.g. artesian well, lake, river.

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