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Front and back covers: Runnymede Bridge across the Thames alongside Lutyens' Bridge (Photos: Harry Sowden)

# Safety factors: Some aspects of CIRIA Research Project 223

Poul Beckmann

What is a safety factor? In the old days when strength of structural elements could usually only be quantified by testing, it was taken to mean the ratio between the failure load under test and the load the element was required to carry in service.

When engineering science had progressed so far as to make engineers believe that stresses could be accurately calculated, the quantification of the safety of the structure took the form of stating that the stresses under service load should be less than the relevant 'permissible working stress.' The permissible working stress was defined as the ultimate strength of the material (when tested in a certain standardized way) divided by a 'factor of safety' which took account of the estimated chance of overload, the variability of the strength of the materials and past design tradition.

Experienced engineers were using this format quite successfully for more than half a century, but the new generation, brought up to respect the words of codes of practice more than the advice of their elders and betters, ran into trouble because some codes had not stated explicitly that certain loads vary independently of others and that variations of materials strengths are of more significance in some cases than in others. This led to the development of 'the partial safety coefficient' approach in which separate factors known as 'partial coefficients' or partial safety factors, were allocated to the loads and to the resistance of the structure.

The first code of practice in Europe to adopt fully this principle was the Danish Foundation Code and generally the Scandinavians seemed to embrace this approach ahead of everybody else.

The European concrete committee introduced the principle in their 'Blue Book' in the 1960s, and, as it is the unwritten rule for any code committee never to be seen to reduce substantially the factor of safety of anything, they set themselves the aim to introduce partial safety factors in such a way that structures designed on the basis of these would provide substantially the same safety as when designed to existing 'permissible stress' codes. They had certain statistical information available which led them to deduce that the factors on the material sought to be in the order of 1.5 for the concrete strength and 1.15 for the strength of the reinforcement. The requirement for similar overall safety then led to a partial safety factor on the loads in the order of 1.5.

The result of this was, however, that the new method was no more rational in terms of the real safety provided than was the old, and hence it was thought that 'something ought to be done about it.' The thought led to CIRIA, in the beginning of 1975, sponsoring a research project aimed at deriving a method by which more rational partial safety factors could be calculated using the tool of the 'reliability theory.'

Reliability can in this connection be defined as the reciprocal of the probability of failure, and this will of course depend on both the loads and the structural resistance and the likely variability of both.

Structural failures do of course mostly occur for reasons other than those which can be connected directly with factors of safety and, whilst it is possible to calculate the risks due to a certain set of factors of safety, the actual risks of a failure of a structure designed to present day codes are about 10 times as big as the calculated risks, due to causes such as gross errors in design and construction and 'acts of God.' (The latter was, incidentally, defined during the research project as something the designer can use as an excuse once. When such an event *has* occurred, future designs should foresee the possibility).

Before getting alarmed about this loose talk of risks, it is however useful to remind oneself of how small the risk of death from structural failure is, compared with a lot of risks which people quite happily accept. (see Table 1) The research project, which was led and co-ordinated by Dr. A. R. Flint, included a lot of work on reliability theory by M. J. Baker of Imperial College, and later on a considerable amount of computer work by Stan Feneron of Building Design Partnership aimed at demonstrating the reliability of a number of structural elements designed to various codes as well as some exercises aimed at showing how the new partial safety factors, derived from probability theory, could be calibrated to give substantially the same overall reliability as would result from present-day codes. Ian Potter of Jenkins & Potter produced a very interesting section on legal and economic aspects of safety.

I was given the task of carrying out a 'state of the art' investigation in current codes and regulations in the UK and abroad. This involved me in studying not only a large number of British standards and codes of practice, but also some of the German DIN Norms, American, French and Danish codes and some reports by the Nordic Committee for Building Regulations. The purpose of this study was to demonstrate the extent to which the partial factors in these codes covered all the variabilities they theoretically ought to allow for.The CEB *Bulletin III* quotes the general design equation as:

$$\gamma_{f_3} \times \text{Effects of } (\gamma_{f_1 D} + \Psi_o \Sigma \gamma_{f_1 L} W_L)$$

$\leq$ Resistance from $\frac{f_k}{\gamma_m}$ where $W_D$ is dead load and $W_1$ covers live loads, wind loads, etc. and $f_k$ is the characteristic material strength.

Both loads and strengths in this equation are entered with a 'characteristic' value, that is a value that has a certain agreed probability of occurring.

$\gamma_m$ is a reduction factor on material strength which is intended to take account of:

(1) *Material strengths occasionally falling below the specified characteristic value:*

The probability of this depends mainly on workmanship and/or quality control. Apart from the masonry code, British codes and drafts don't allow for this. The probability of a failure depends, everything else being equal, not only on the characteristic strength of the material but also on the variability of the strength. High mean strength and high variability gives greater risk for the same characteristic strength. The only mention of this is found in the NKB document.

(2) *Possible differences between strength of the material in the structure and that determined from control test specimens:*

This difference depends of the frequency of sampling related to the consistency of production, and on the different behaviour of the standard sample in the testing machine from that of the bulk of the material in a real structure. For example, the British Standard Concrete Cube is compacted and cured in a completely different way from the concrete in the structure.

It follows that when one is assessing an existing structure on the basis of tests of actual strengths of material in the structure $\gamma_m$ can be reduced.

(3) *Possible weakness in the structural material resulting from the construction process:*

This is a function of workmanship and supervision. British codes and drafts, except the masonry draft which varies $\gamma_m$ with manufacturing quality control and site testing, follow American and Canadian practice in having invariant $\gamma_m$s. Scandinavian documents grade their materials factors according to frequency of acceptance testing and site control.

(4) *Possible inaccuracy in the assessment of the resistance of a structural element resulting from modelling errors:*

It is impractical in a code of practice to formulate simple calculations which will predict accurately failure for the whole range of each type of member that has to be covered by the code. Hence, some allowance has to be made for the resulting errors. Grossly erroneous assumptions can however *not* be covered by partial safety factors.

(5) *The effects of poor dimensional accuracy in the finished structure on the resistance of a section:*

Inaccuracies in construction are a function of workmanship and hence supervision. The British draft masonry code and some Scandinavian codes graduate $\gamma_m$ accordingly, but they are the only ones to do so. The effect of inaccuracies is greater for small members, so $\gamma_m$ should vary ideally with size, but no code seems to cater for this.

$\gamma_{f_1}$ *is a multiplier applied to loads or imposed deformations to take account of the possibility of loads exceeding their characteristic value:*

There is here the difficulty that very few types of load are sufficiently recorded to establish a mean value, let alone a characteristic. The codes tend to apply factors to nominal 'statutory' loads traditionally stipulated in regulations.

Certain loads, eg: water pressures, are calculable with less uncertainty than usual live loads and therefore deserve a lower value of $\gamma_f$. Some codes allow this, others don't.

**Fig. 1 ▶**
Typical relationship between cost and total probability of failure

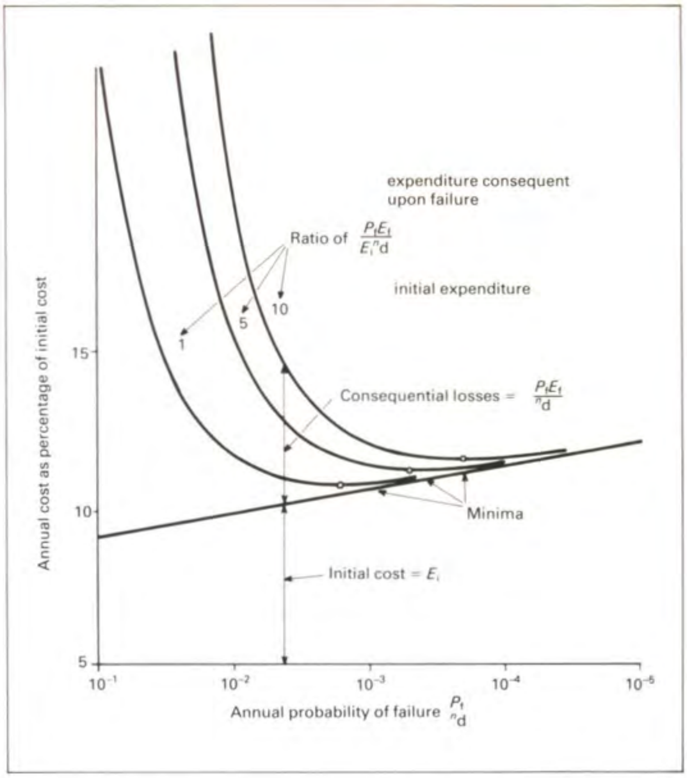



**Fig. 2 ▼**
'Naive' application of partial safety factors to the problem of stability defeats the object

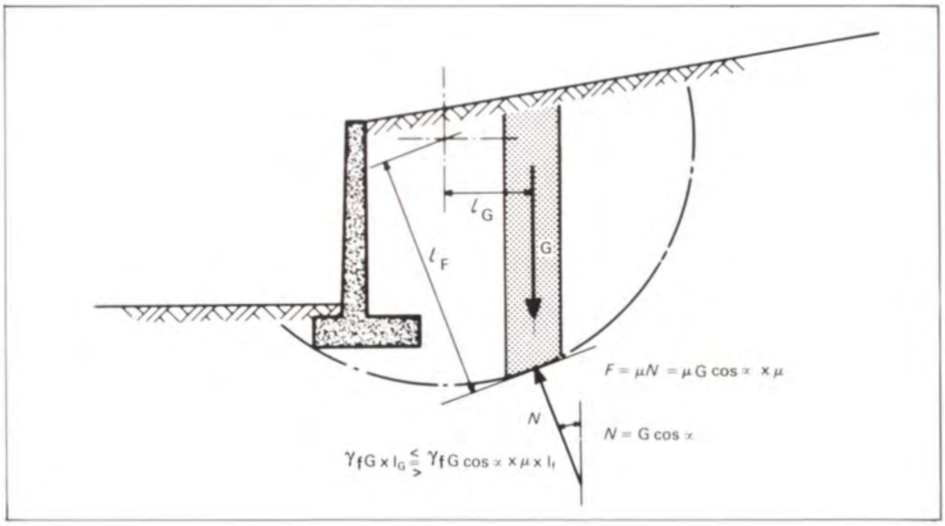



**Table 1 :** Comparative annual probability of death per 10 000 persons

| | Hours exposure/ annum | Annual risk/10,000 persons | Approximate annual risk/person |
|---|---|---|---|
| Mountaineering (International) | 100 | 27 | $10^{-2}$ |
| Distant water trawling (1958-72) | 2900 | 17 | |
| Air travel (crew) | 1000 | 12 | $10^{-3}$ |
| Coal mining | 1600 | 3.3 | |
| Car travel | 400 | 2.2 | $2 \times 10^{-4}$ |
| Construction site | 2200 | 1.7 | |
| Air travel (passenger) | 100 | 1.2 | |
| Home accidents (all persons) | 5500 | 1.1 | $10^{-4}$ |
| Home accidents (able bodied) | 5500 | 0.4 | $4 \times 10^{-5}$ |
| Manufacturing | 2000 | 0.4 | |
| STRUCTURAL FAILURE | 5500 | 0.001 | $10^{-7}$ |
| All causes (England and Wales, 1960-1962) | | | |
| Male age 30 | 8700 | 13 | $10^{-3}$ |
| Female age 30 | 8700 | 11 | |
| Male age 50 | 8700 | 73 | |
| Female age 50 | 8700 | 44 | |
| Male age 53 | 8700 | 100 | $10^{-2}$ |

$\Psi_0$ *is a multiplier applied to load combinations to take account of the reduced probability of all loads exceeding their characteristic value simultaneously:*

Some codes, e.g: the Canadian, apply this as a multiplier to the sum of $\gamma_{f_1}$ factored loads, others vary the $\gamma_f$ for each type of load depending on the combination.

$\gamma_{f_3}$ *is a multiplier on load effects to take account of errors in predicting load effects as a result of neglecting dimensional inaccuracies: to take account of non-linear behaviour where there is a magnification of load effects:*

Different accuracies of assumptions and computations should qualify for correspondingly different values. In fact, the accuracy assumed in the implied $\gamma_{f_3}$ (as incorporated in the various codes' $\gamma_f$ – values) is nowhere defined. Non-linear behaviour is another matter and ought not to be introduced here, as its effects vary so much from case to case.

$\gamma_n$ *does not appear in the equation but is used to modify* $\gamma_f$ *or* $\gamma_m$ *to take account of the nature of the structure and its behaviour and the seriousness of attaining the limit state:*

Elements and structures that fail suddenly should have a greater overall factor of safety than those which, due to their ductile behaviour give warning of collapse. The German, American and Canadian codes seem to give the most explicit allowance for this distinction. None deal with hidden ductile structures, where the warning signs cannot be observed.

Structures, the collapse of which may kill or injure a large number of people, would seem to qualify for higher safety standards than a lintel over a door in a transformer house. The Scandinavian drafts have three classes of structure according to the probability of death and injury and the level of damage to property resulting in case of collapse. The Canadian code has an 'importance factor' but only with two values, the lower one applying to certain farm buildings.

Difficulties are sometimes experienced in allocating some of the partial factors, notably $\gamma_{f_3}$ to one or the other side of the equation. A factor of 1.4 to be applied to own weight is, in the absence of any explanation, taken by some engineers as an insult to their standards of calculation and the application in *CP110* of 1.4 and 1.0 to own weight in adjacent spans, stretches the imagination of most of us. The explanation lies of course in the positioning of $\gamma_{f_3}$.

Certain earth stability problems throw up even worse predicaments: in the classical slip circle calculation the calculated factor of safety remains constant regardless of the value chosen for $\gamma_f$.

Apart from the partial safety factors there are certain other parameters that affect the reliability of a structure:

### Design life

The limit state philosophy usually includes the phrase: 'the structure shall not become unfit for its intended use during its design life.' No code defines the design life and yet it would seem essential to do so when dealing with materials with strengths which reduce with time such as timber or forces which increase with age such as drag on marine structures due to barnacle growth. Some confusion also occasionally occurs between design life and return period of natural phenomena, such as wind forces.

### Characteristic strength and materials testing:

The 'characteristic' value of the strength of a material is defined in British and CEB practice as being the one attained or exceeded by 95% of a theoretically infinitely large number of tests, and there are statistical methods for calculating this value for a finite number of tests. Whether these methods are adequate to deal with the testing frequency in *BS 4360*: one tensile test for every 40 tonnes, is another matter. (It corresponds to one test for each km length of 12in. × 6½in. UB (305 × 165 × 40) or one test per 2½ miles of 4in. × 3in. angle (102 × 76).)

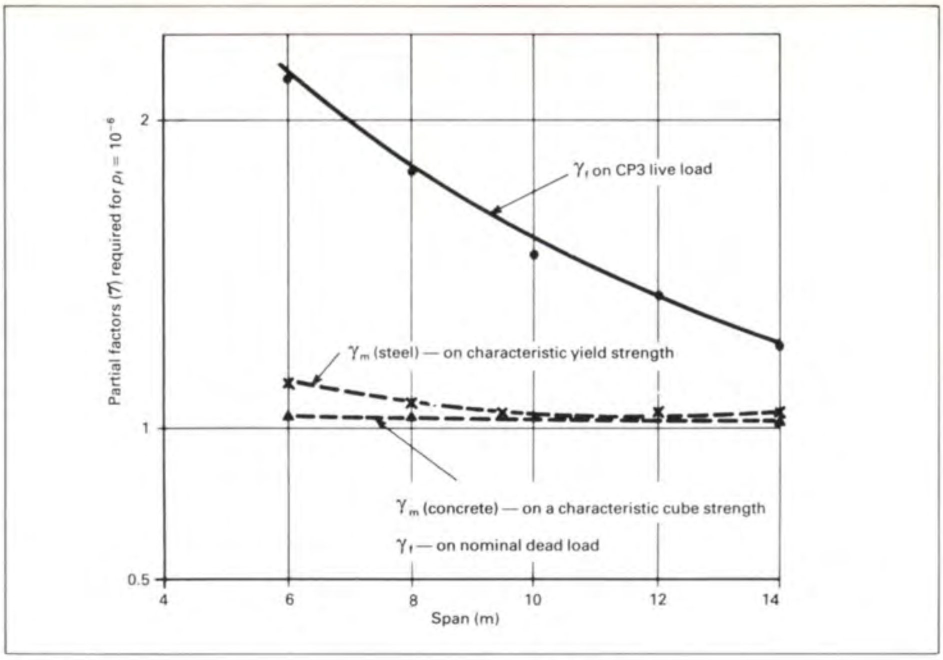
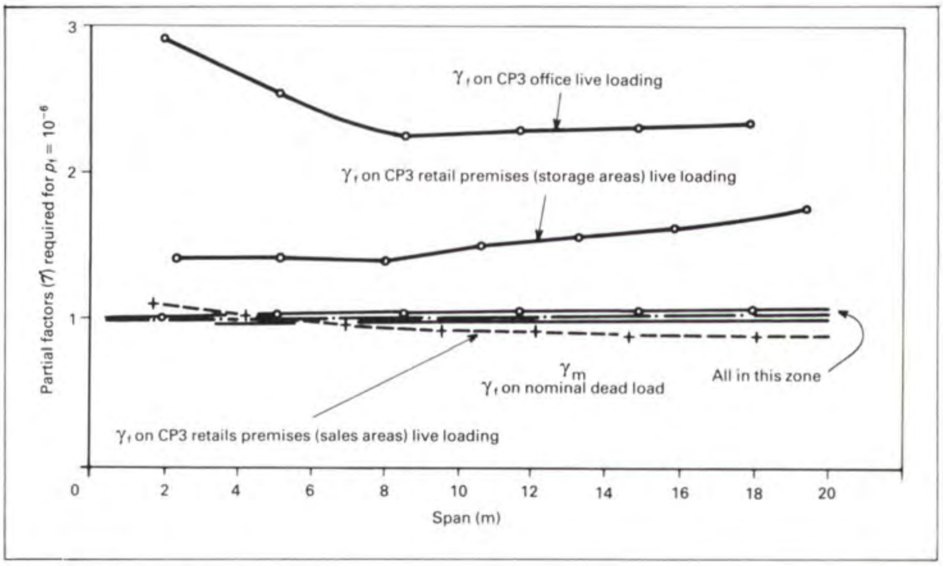



**Fig. 3 ▲**
Partial safety factors for singly-reinforced beams—office loading

**Fig. 4 ▼**
Partial safety factors for Grades 50 universal beams




### Definition of failure:

Mention was made earlier of modelling errors in calculation of resistance of a structural member, but occasionally our traditional assumptions about the mode of failure lead to errors not of plus or minus 15% but several hundred %. For instance, a beam spanning between heavy columns at 1st floor level of a multi-storey building will carry, in a catenary action, a load several times that which will collapse an identical beam at the top of the building.

### Design practice in allied fields of engineering:

The treatment of safety in aeronautical, marine and nuclear engineering was studied. The information obtained on nuclear practice was scanty and did not indicate any methods which were more rational than current structural practice. Aeronautical engineering uses probability theory extensively to predict design loads but has to apply severe modification factors to make the results compatible with the safety records of previous successful models. The same applies to shipbuilding. (Probability theory would have predicted B707s falling out of the skies by the dozens!) In both instances we found little that could usefully be adopted in building and civil engineering design because (a) we do not have enough data on performance, failures and their causes, (b) we do not have long enough production runs to be able to benefit from prototype testing, and (c) we cannot practically enforce a materials testing regime anywhere as stringent as the aircraft industry. This article only deals with a small part of the entire research project which is fully described in CIRIA report no. 63.

I hope that none of the hardworking chairmen and members of code committees take any of the comments in the report or in this article as a criticism of their work. Having been involved on the sidelines of one of those committees, I know that it is difficult just to make the transition from permissible stresses to partial safety factors, especially when it is also stipulated that design according to the new principle must not result in something significantly different from the end result of the old code.


### Acknowledgement

Figs 2, 3 and 4 are reprinted from:
CIRIA Report 63, Rationalisation of safety and serviceability factors in structural codes 1976.

# Runnymede Bridge

## Bill Smyth

The two bridges over the Thames at Runnymede allow an interesting comparison to be made. One was designed by an architect with help from an engineer and the other by engineers with help from architects.

Sir Edwin Lutyens designed the first bridge to carry the Staines Bypass. The conceptual design was completed in 1939, he died in 1944 and the bridge was built in 1961/2. It is basically a thin arch of white concrete spanning 56 m over the river and thrusting against cellular abutments founded in London clay. In the space between arch and deck are steel trusses transferring the loads to the arch and stiffening it. The bridge was constructed by cantilevering the trusses out from the abutments, and suspending the formwork for the arch from them. The spandrels are closed by brick panels which conceal the trusses, the abutments are clad with brick and the parapets are Palladian balustrades of Portland stone. There are two towpath arches through the abutments and these are lined with brickwork and have Portland stone facings.

The Lutyens bridge was designed for six lanes of traffic and two footways. The M25 is now to cross the Thames and requires four lanes and two hard shoulders in each direction and these are sandwiched between two lanes of the A30 and a footpath in each direction, so that a bridge of twice the original width is required.

There were two major problems, one aesthetic and one technical which faced the design team of the Civil Engineering Division and Arup Associates.

The first is what kind of bridge can you possibly put alongside the Lutyens bridge? The Department of the Environment (now Transport) had emphasized the importance which they attached to the problem by appointing Arup Associates directly as consulting architects. The second is that Lutyens' bridge is founded in London clay which is a bad material for founding an arch in. Some spreading of the abutments was allowed for in design but not enough to cope with ground movements caused by another arch alongside, and we had to be careful not to break its back.

We made a cardboard model of the Lutyens bridge and we used this in conjunction with rough models made from card and balsa to examine the visual effects of the various possibilities. The modelscope was a great help because it allowed us to get the viewpoint of a scale-sized man. We decided to try four kinds of schemes. The first was extension of the existing structure. This was thoroughly unsatisfactory and we were able to get the roads realigned slightly so as to create a gap of 3 m between the older and newer bridges.

The other groups of schemes were:

(a) Imitations of Lutyens' bridge in appearance or form only

(b) Neutral bridges (a very thin flat plate for instance)

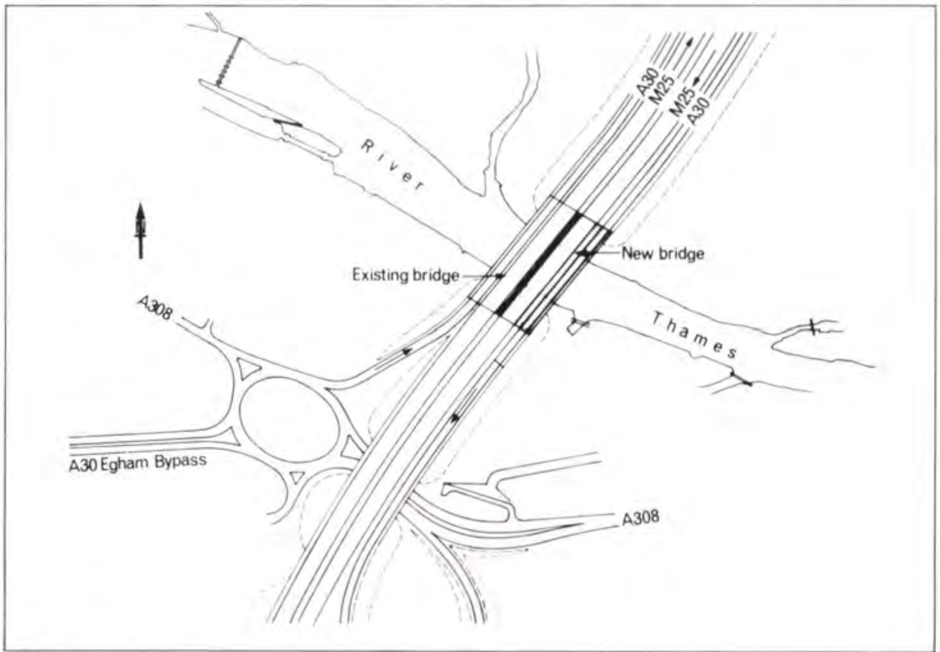
(c) Other kinds of bridges with curved soffits which would be compatible with the existing bridge.

The final result of the studies of structure, cost, and appearance was our project design which was completed in 1972.

The bridge deck is carried by four concrete frames whose soffits over the river lie on the same surface as that of Lutyens' river arch. Each frame is made up of two balanced half-frames which are connected over the centre of the river. The trapezoidal portal ring at the heart of each half-frame is supported by two sets of laminated rubber bearings and a simple truss cantilevers out on each side of the portal. The top members of the frames are prestressed and the other members simply reinforced. Foundation loads (which are of course predominantly vertical) are taken into the clay by large diameter bored piles with underreams.

The bridge superstructure is made of white concrete using Balidon aggregate and white cement, and is bush-hammered. A similar, though less strong, concrete was used by Arup Associates at St. John's College, Oxford.

The bridge deck is made up to the correct level with lightweight concrete, which helps to reduce the effect of the very onerous current requirements for differential temperature, with waterproofing and black top surfacing. The parapets are precast with the same finish as the rest of the bridge.

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2▼

3▼

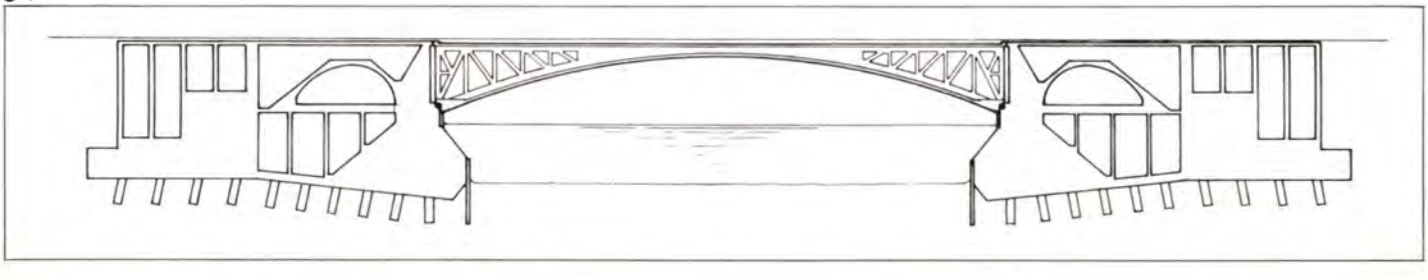


**Fig. 1**
Site plan

**Fig. 2**
Lutyens' bridge
(Photo: R. Benaim)

**Fig. 3**
Longitudinal section through Lutyens' bridge

The superstructure was detailed so that it could either be constructed in situ on falsework (on piles in the river) or by casting the frames on the banks and sliding them to their final positions. The bridge was built by Fairclough who decided to use the sliding method.

The contractor had two sets of forms, one for each side of the river, so he got four uses of them. The form and falsework were arranged so that after casting and the first stage of stressing, the units could be jacked clear of the soffit shutter and slid sideways out of it on PTFE coated skids sliding on tracks which consisted of a thin layer of stainless steel fixed to mild steel plates on a concrete beam supported by piles. After the sideways move, the unit was jacked up and the skids and track rearranged so that it could be slid forward to the river bank, again jacked and slid sideways to its final position. The motive force for sliding was produced by hydraulic jacks pushing between the frame and nuts on *Dividag* bars or, in some cases, using strands and automatically gripping jaws on the jacks.

When the two half-frames of each pair were finally positioned there was a 2m gap between them. The second stage cables were placed, the gap concreted and the second stage stressing took place. The top member of each frame had sockets cast into the sides, and rails were fixed to these which carried the travelling forms for the deck. When the deck had been concreted, the third stage cables could be threaded and stressed. The contractor chose to use the BBRV 55-wire system, so the wires forming those cables which were threaded had to be button-headed on site.

At the time of writing, work on deck finishes and parapets is going on. When our bridge is ready, traffic will be diverted onto it and work on strengthening Lutyens' bridge can start.

**Credits**

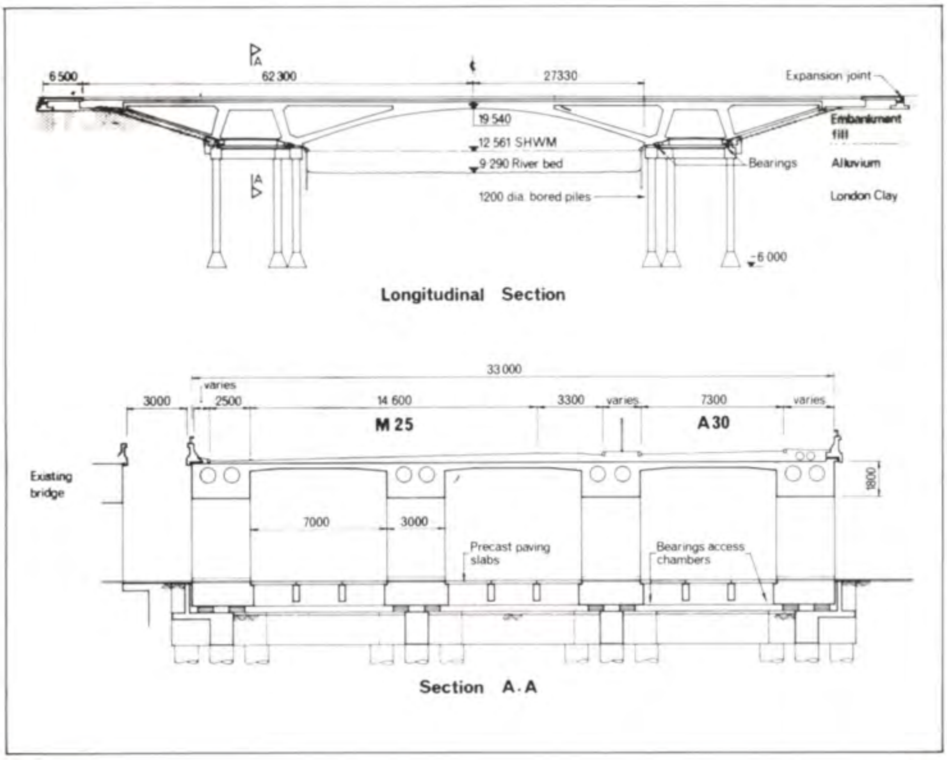
*Client:*
Department of Transport
South Eastern Road Construction Unit

*Road design:*
South Eastern Road Construction Unit
Surrey Sub Unit

*Bridge design:*
Ove Arup & Partners, Civil Engineering Division with Arup Associates as consulting architects

*Contractor:*
Fairclough Civil Engineering (Southern) Ltd.

Lutyens' bridge was designed in concept by him with Mr. Fitzsimons as engineer. The works design was carried out by C. W. Glover & Partners, consulting engineers and Mr. George Stewart, consulting architect.

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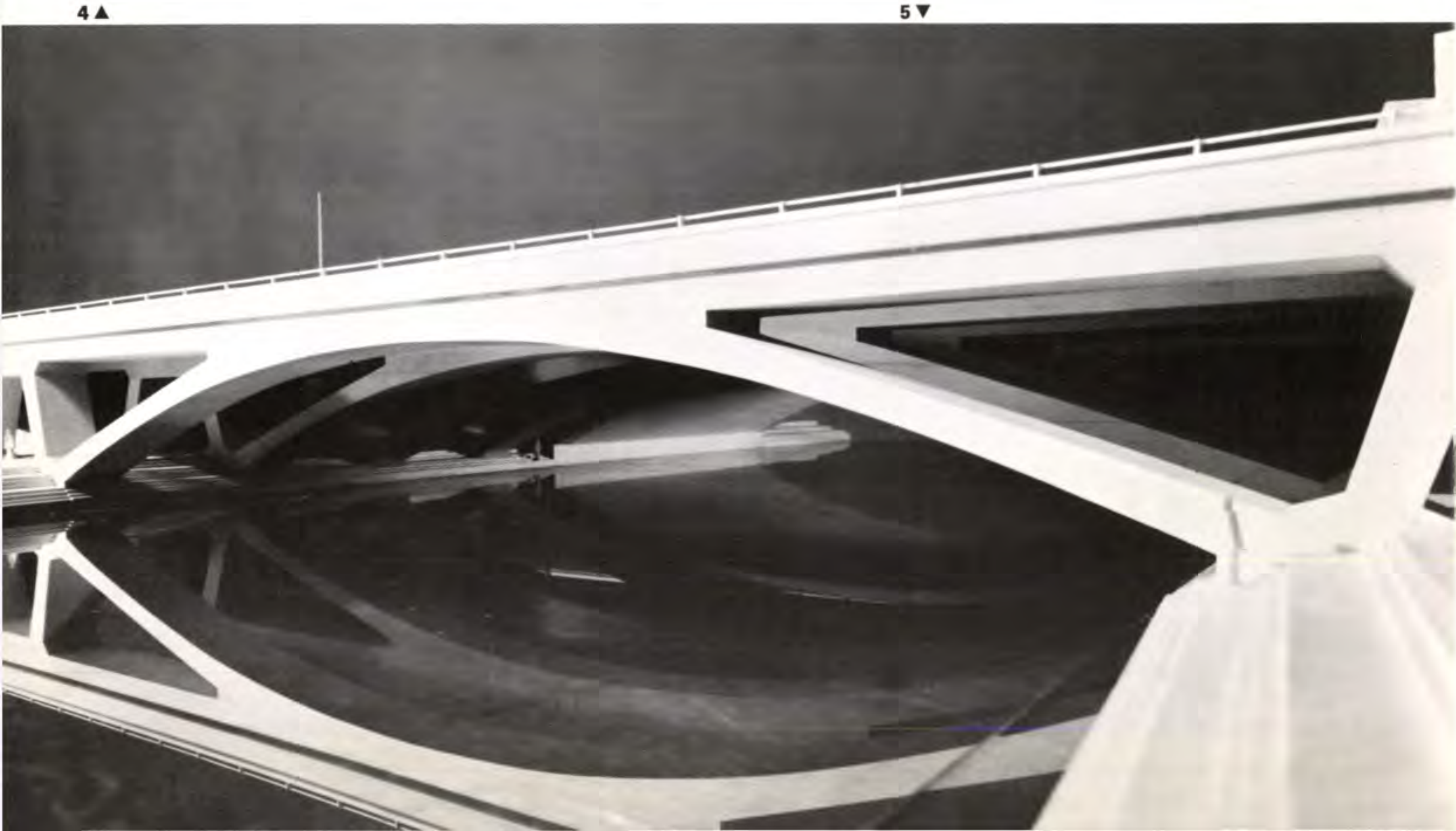


**Fig. 4**
Longitudinal section and cross-section of Arups' bridge

**Fig. 5**
Model prepared for submission to the Royal Fine Art Commission. (Photo : Henk Snock)

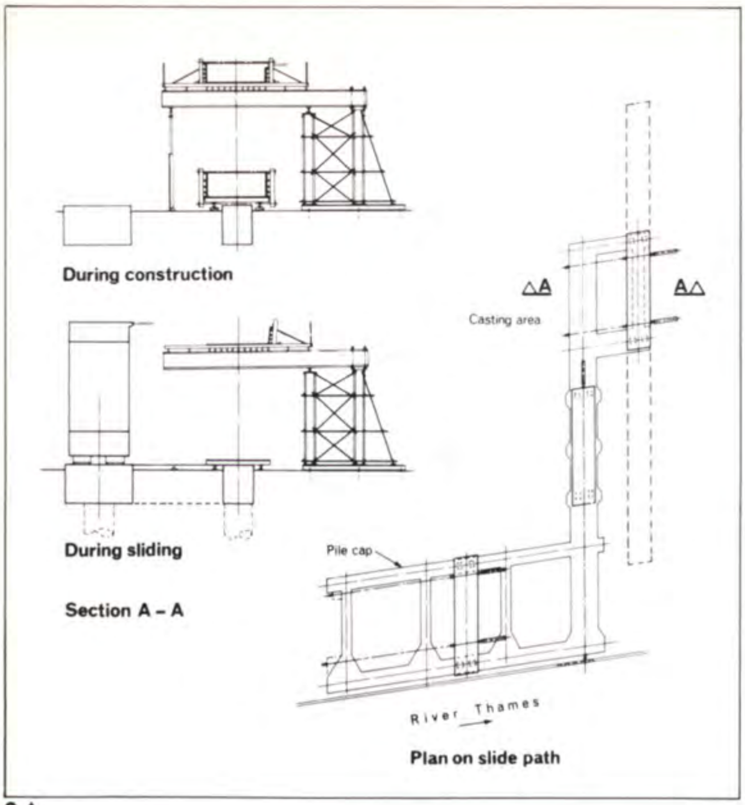
**Fig. 6**
Diagram of sliding

**Fig. 7**
View from South bank
(Photo : Handford Photography)

**Fig. 8**
The deck shuttering is carried on rails bolted to the sides of the frames
(Photo : Kodak Vision)

**Fig. 9**
Aerial view looking north
(Photo : Milligan)

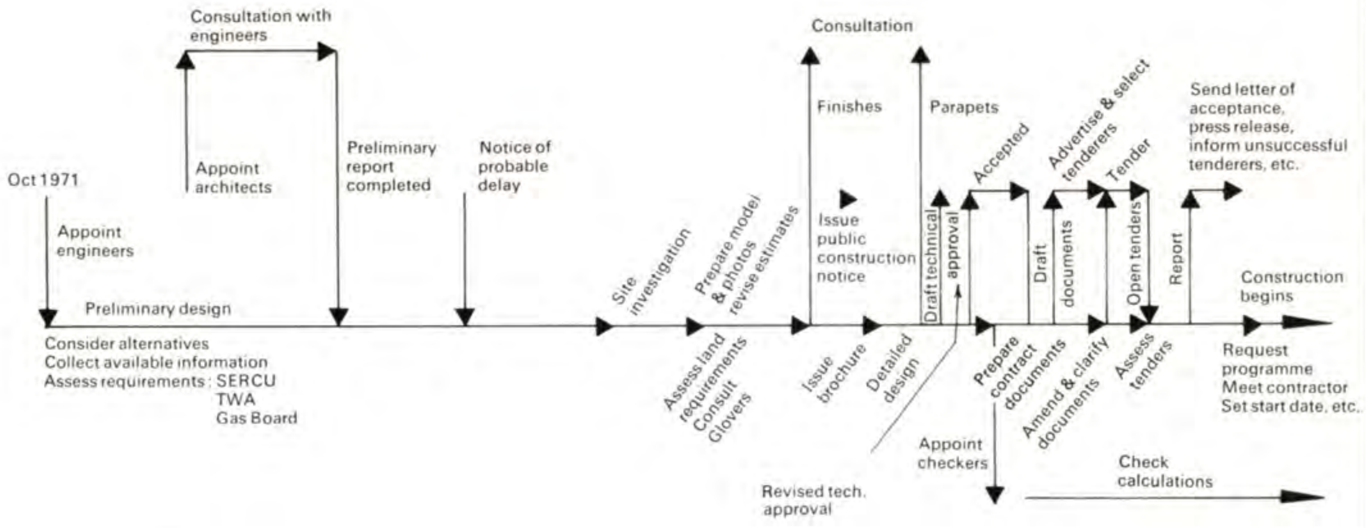
**Fig. 10**
General view of new bridge
(Photo : Harry Sowden)


During construction
During sliding
Section A – A
Casting area
Pile cap
River Thames
Plan on slide path
6 ▲

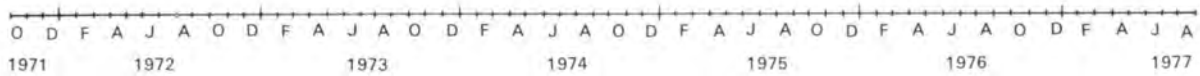

8 ▲

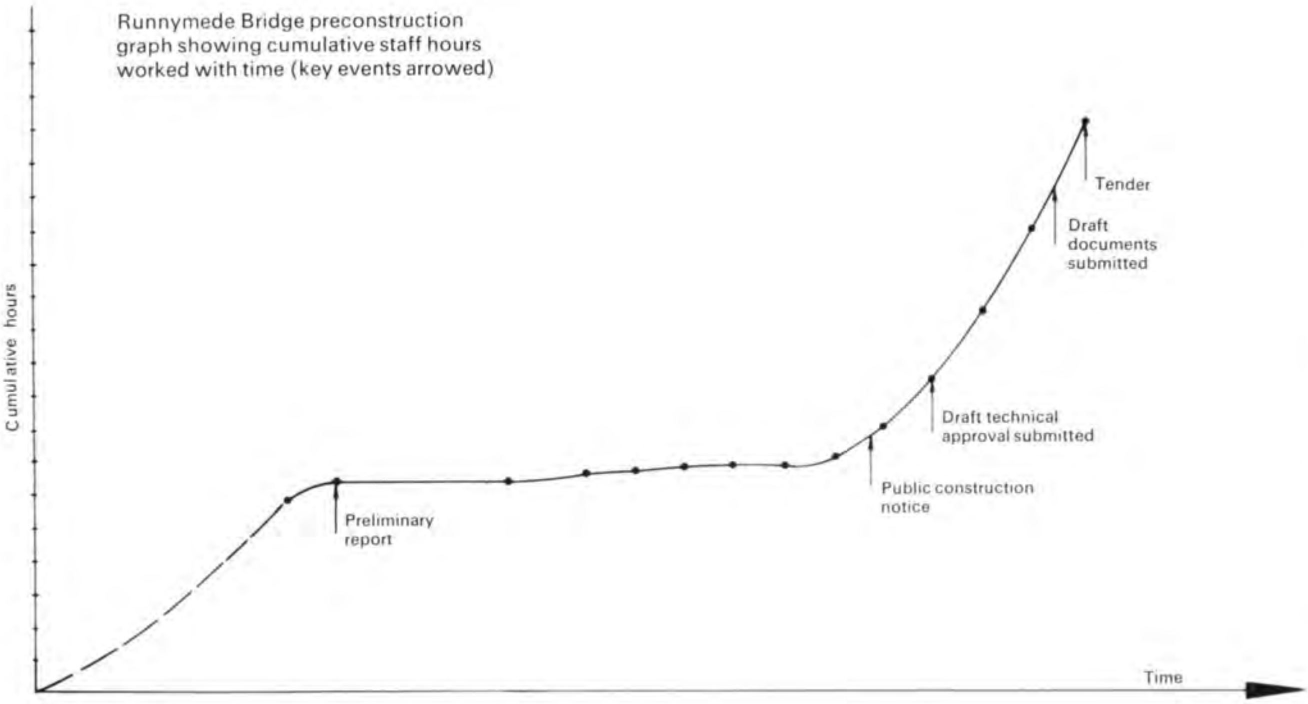

7 ▲


9 ▲




Simplified, time-scale precedence diagram
Runnymede Bridge preconstruction stages




**Runnymede Bridge**

Method study diagrams by Greg Adamiw

There is an interesting sidelight on our work on Runnymede Bridge. Lancaster University runs a post-graduate course in Civil Engineering Production Studies. Part of it involves the students in spending two weeks at each of a number of different organizations investigating some aspect of the running of jobs. Arups had two of the students in February 1979 and one of them, Greg Adamiw, studied our method of working on the Runnymede Bridge project, talking to people and studying files, time sheets, operating plans and other documents. Greg's very interesting report included a precedence diagram and a diagram showing the total number of man hours Arup staff booked to the project plotted against time. These diagrams are reproduced here. They show quite clearly the considerable gap between project design and works design which has been a feature of many of our highway and bridge jobs.

# The south abutment at Kessock Bridge, Scotland

Ken Cole

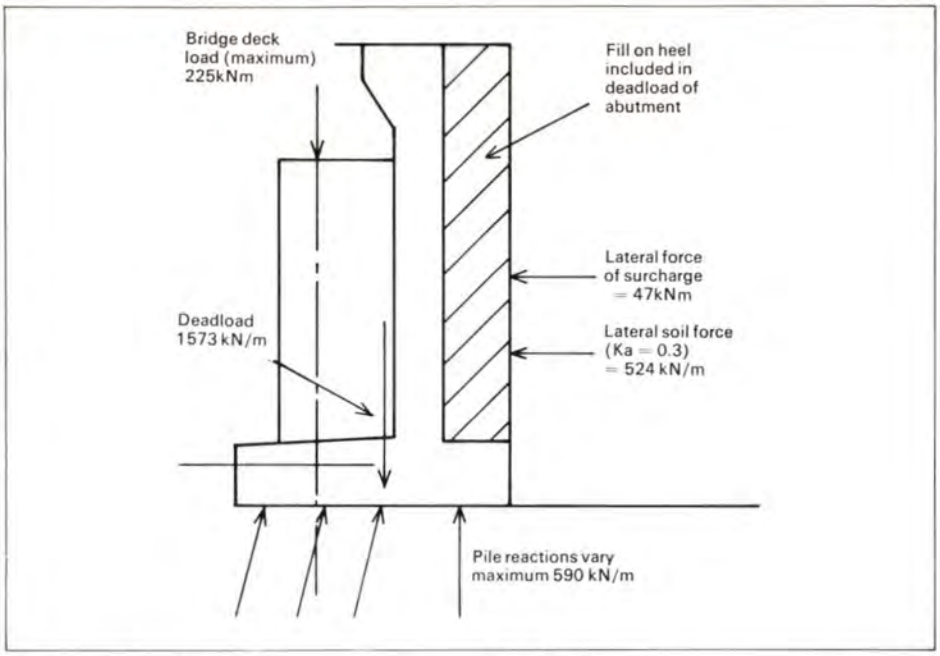
## INTRODUCTION

The Kessock Bridge when completed will span the Beauly Firth at Inverness, Scotland. As shown in Fig. 2, the abutment to the southern approach viaduct retains a selected granular fill embankment some 12m high, and supports one end of the first 64m span. The bearings of the viaduct are designed to transmit vertical load only, longitudinal forces being absorbed in the piers and at the northern abutment.

## DESIGN OF FOUNDATIONS

### Soil conditions

The great depth of alluvial deposits at the site have influenced the choice of foundations; a borehole for a pier adjacent to the navigation channel reached 95m below bed level, and a borehole 1 km away reached 89m below ground level, both encountering only alluvial deposits. The alluvial deposits increase in density with depth and pad foundations have been adopted where maximum bearing pressures can be kept economically to within 250 kPa. Where bearing pressures exceed this, or the depth of construction below water exceeds 10.5m, steel H-section bearing piles have been driven into the alluvial deposits.

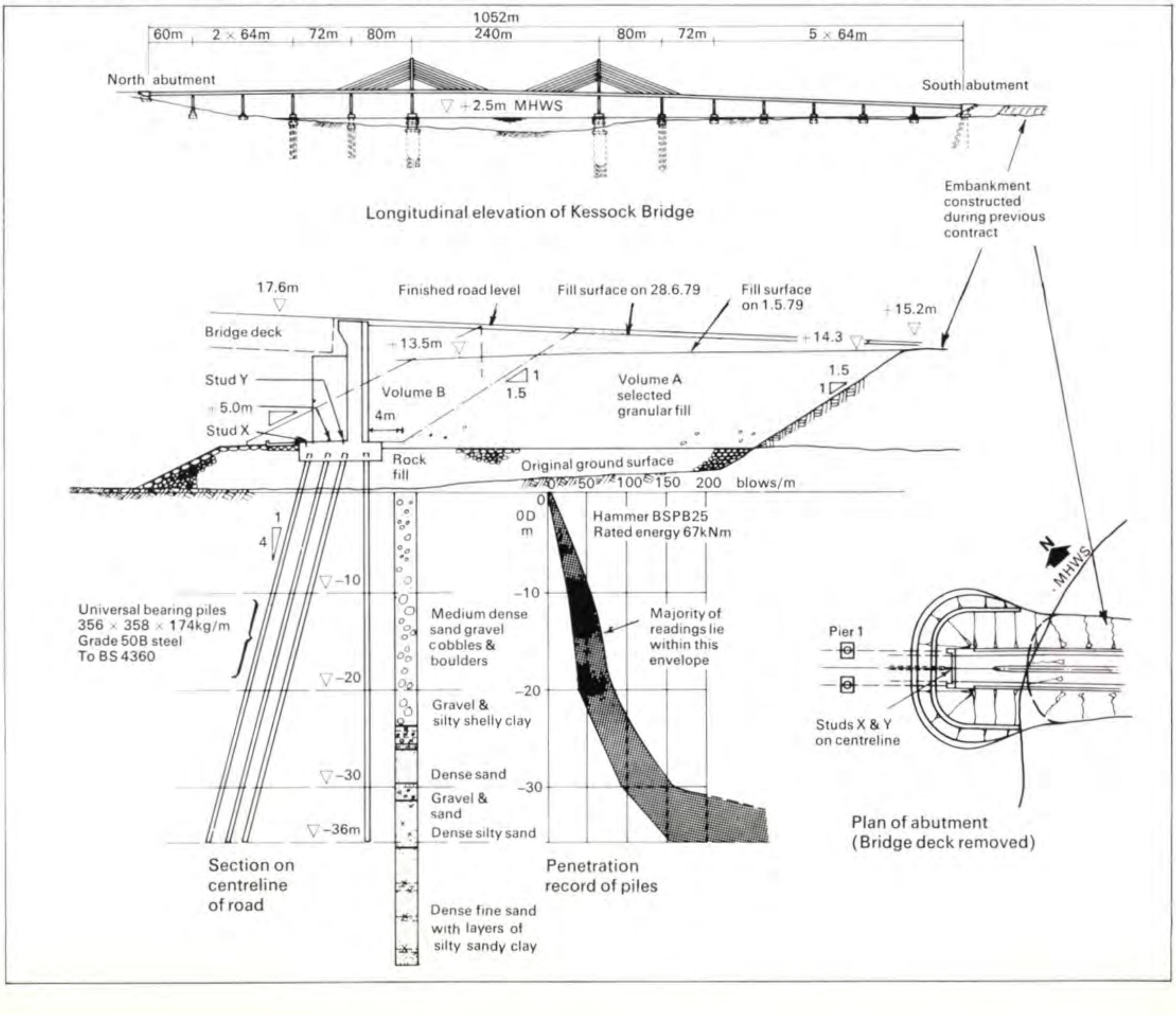



### Abutment foundations

Conventional analysis of the southern abutment foundation indicated that three rows of piles raked at 1 in 4, and a rear row of vertical piles were required to provide adequate support for the loads shown in Fig.1. The driving record of piles is given in Fig. 2; test loading of a vertical pile gave 7mm settlement at the design working load of 1600 kN.

**Fig. 1**
Loading diagram of forces on abutment (non-seismic loads)

**Fig. 2**
Location and general details



## MOVEMENT OF THE FOUNDATIONS

### Analysis treating only forces on abutment

Analysis of the movement of the abutment, taking account of the forces shown in Fig. 1, and treating the piled foundation as elastic springs, showed that abutment movements would tend to be downwards and forwards, the base of the abutment moving some 40mm towards the north. However, it was evident that the analysis of movement of the abutment could not be considered in isolation from the ground movements that would be generated by the construction of the short length of granular fill embankment between the already placed embankment and the abutment.

### Loading sequence

The loading sequence used is shown in Fig. 4, the construction process being simulated by four stages, A to D. To check on the effect of the simplifying assumption that the embankment load applied only a vertical load, Stage Z, in which horizontal shear loads from the embankment were applied, was run in addition to Stage B in Analysis 3.

### Analyses of interaction of soil and structure

A finite element program, SAFE,[1] incorporating facilities to take account of soil and structural material properties, was used to compute ground movements and stresses in the structural elements. Previous computations using SAFE program, including modelling of anchored retaining walls, had shown that the program gave reasonably accurate predictions of movements when compared with field measurements.

A two-dimensional finite element model of the abutment and the soil was established, see Fig. 3, and a series of analyses undertaken, the assumed soil and structural material properties for Analyses 3, 4 and 5 being shown in Table 1.

The soils were given linear elastic properties and assumed to be drained, since their rate of consolidation was expected to be rapid. The assumptions concerning the pile elements are stated in Fig. 5; for Analysis 4 the piles were assumed to be absent in order to demonstrate the general validity of the method. The main defect of the model was that the pile elements had to be considered as having the same unit 'depth' (of 1m) as the soil and concrete elements, thus prohibiting relative movement in the line of the bridge between the piles and adjacent soil.

### Computed and measured movements

The results of the computations of movement are presented in diagrammatic form in Fig. 5, with the measured results to date shown on Analysis 3. Studs X and Y, on which level readings have been made, are located on the road centreline at the positions shown in Fig. 2. Site measurements of movement of the abutment in the line of the road were attempted, but, showing little if any movement and being difficult to make with sufficient accuracy, they were discontinued. The abutment movements have therefore been presented on the assumption that only vertical movement took place.

### Movements were time dependent

The diagram giving settlement against time, Fig. 6, shows that realization of full settlement at each stage of loading was delayed by classical consolidation involving the dissipation of excess pore water pressures from the fine grained soils, complete dissipation for each stage taking about 6 months. The specified sequence of embankment filling was that Volume A should be completely placed in Stage B (Fig. 4) before the placing of Volume B was started in Stage C, the object being to eliminate the pressure of fill material against the stem of the retaining wall as the base took up the major part of its backward rotation during the placing of Volume A, see Fig. 5. In the event, the fill surface was raised uniformly over the entire fill length from about 5m above datum to 13m above datum in one continuous lift, followed by further filling to road base level at 17m above datum after a delay of two months.

**Table 1**
SAFE finite element input data of material parameters. All strength and modulus properties in $MN/m^2$.

| Material parameters | Soil (Analyses 3 & 4) | | Concrete (All analyses) | Piles (Analyses 3 & 5) | | Soils (Analysis 5) | |
|---|---|---|---|---|---|---|---|
| Material number | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Material type | LE | LE | LE | LE | LE | LE(s) | LE(s) |
| Drainage | Drained | Drained | Drained | Drained | Drained | Drained | Drained |
| $E_1$ | 50 | 30 | 30 000 | 50 | 50 | 50 | 30 |
| $\upsilon_{1z}$ | 0.3 | 0.3 | 0.1 | 0.3 | 0.3 | 0.3 | 0.3 |
| $E_2$ | 50 | 30 | 30 000 | 4 809 | 7 642 | 50 | 30 |
| $\upsilon_{12}$ | 0.3 | 0.3 | 0.1 | 0.3 | 0.3 | 0.3 | 0.3 |
| $G_{12}$ | 19.20 | 11.54 | 13 636 | 19.20 | 19.20 | 19.20 | 11.54 |
| $\infty$ | 0 | 0 | 0 | −14.036 | 0 | 0 | 0 |
| $c'$ | — | — | — | — | — | 0.017 | 0.017 |
| $\phi'$ | — | — | — | — | — | 0 | 0 |

**Note**

| | | |
|---|---|---|
| LE | = | linear elastic model |
| LE(s) | = | linear elastic model with strength limitation |
| Drained | = | excess pore water pressures assumed dissipated |
| $E_1$, $E_2$ | = | principal Young's modulii in x, y plane, suffix 1 denoting horizontal (in plane), 2 denoting vertical and 3 denoting horizontal (normal to plane) directions when $\infty = 0$ |
| $G_{12}$ | = | shear modulus between directions 1 and 2 |
| $\infty$ | = | anticlockwise inclination of direction 1 to horizontal (degrees) |
| $c'$ | = | cohesive resistance (limiting value) |
| $\phi'$ | = | frictional resistance (limiting value) |

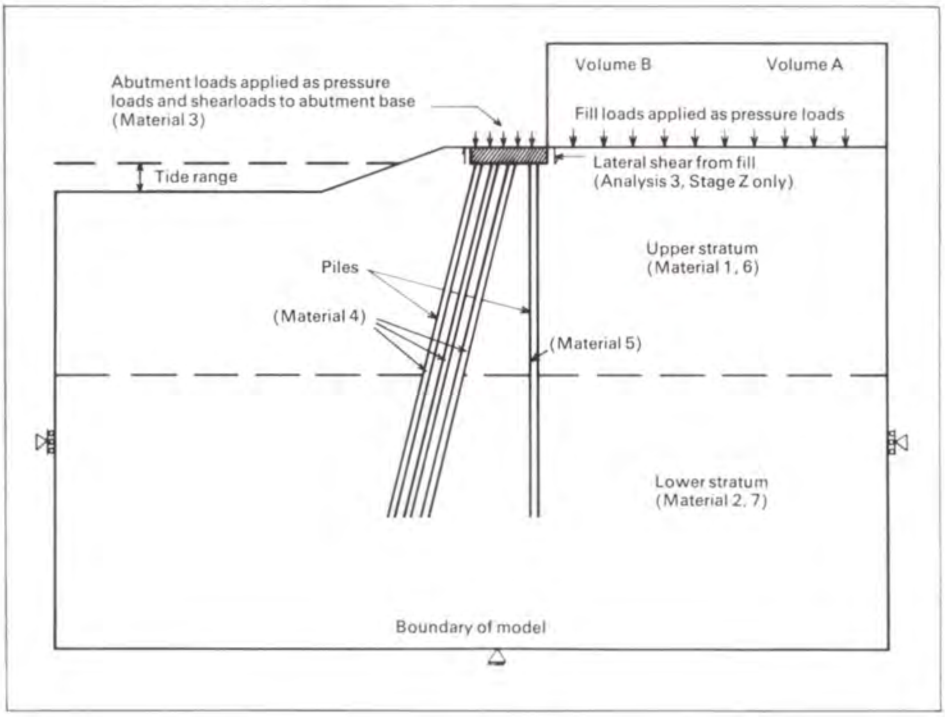



**Fig. 3**
Finite element model.
For properties of materials 1 to 7 see Table 1

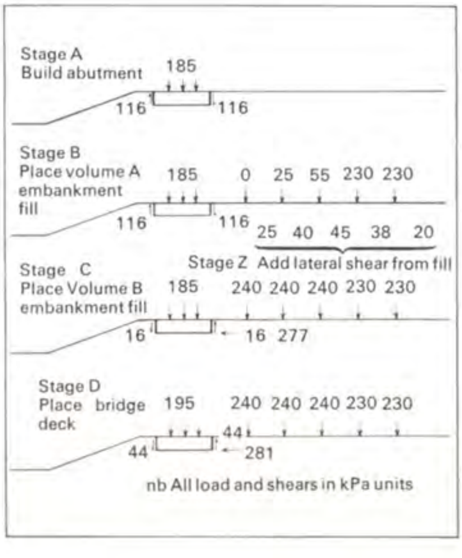



**Fig. 4**
Loading diagram for finite element model

## EFFECT OF MOVEMENTS ON STRUCTURE

### Restraint on backward rotation

As a consequence of following the described sequence of filling, the lateral pressure of retained soil restrained the tendency of the base to tilt backwards by developing the bending strength of the connection between stem and base, the measured average angular rotation between the stem and base being about 1 in 900 after the filling to 13m above datum. During the subsequent filling this rotation increased to about 1 in 525. It is thought that the average vertical movements of base will have not been affected by the difference between the intended and the actual sequences of filling, but the backward tilt of the base may have been reduced.

### Stiffening effect of piled foundation

From Figs. 5 and 6, reasonably good correlations are apparent between the predicted settlement from Analysis 3 and the actual settlement on 17 January 1979, when the loading was the abutment base and stem (Stage A) and on 18 June 1979 when all loads except the bridge deck had been placed and consolidation under those loads was virtually complete (Stage C). It is evident that the piles moved together with the surrounding soil to give much greater settlements than could have been predicted from the results of the pile test loading and considerations of group action; the movements were of the same form but smaller than those that would have occured if the foundation had not had piles. The smaller actual backward tilt than predicted by Analysis 5 indicates that the soil adjacent to the rear row of piles did not move differentially to the piles, and that shear failure played little part in the overall behaviour.

### Credits

The client is the Scottish Development Department who kindly gave their consent to the publication of this paper.

The contractor is a consortium of Cleveland Bridge and Engineering Company and Redpath Dorman Long (Contracting) Ltd. The bridge was designed by Dr. Ing H. Homberg in association with Cleveland, and the substructure by Trafalgar House Engineering Services in association with RDL.

The joint engineers for the client are Crouch and Hogg of Glasgow and Ove Arup and Partners.

Acknowledgement is made of the dedication of the site staff in obtaining the readings which made possible this contribution.

### Reference

SIMPSON, B., O'RIORDAN, N. J., *and* CROFT, D. D. A computer model for the analysis of ground movements in London Clay, *Geotechnique, 29* (2), pp. 149-175, 1979.

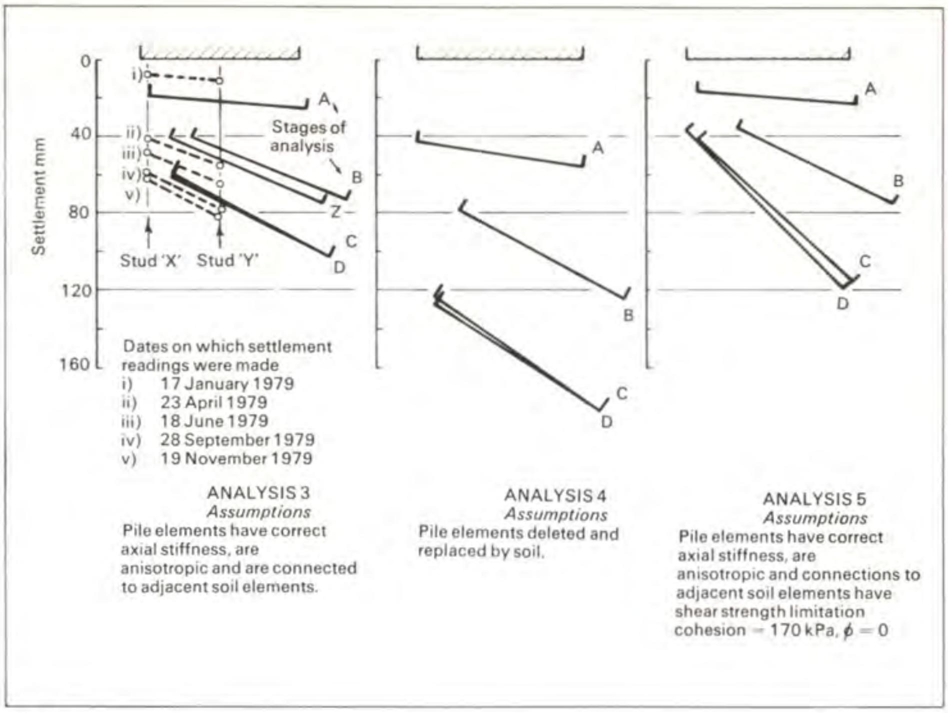
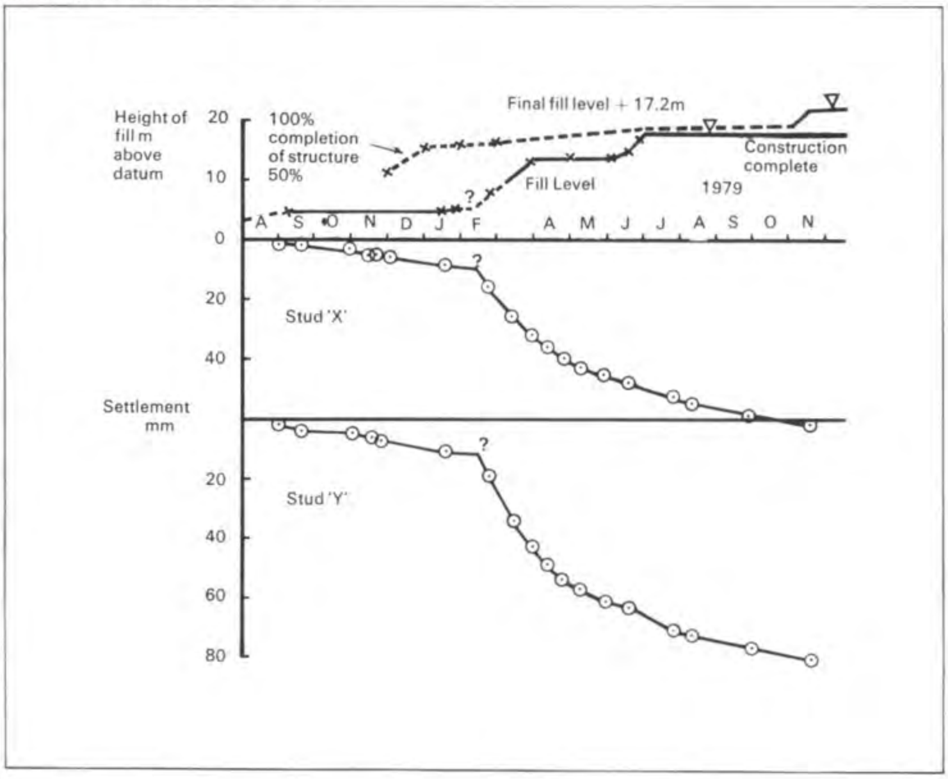



**Fig. 5**
Computed and measured movements

**Fig. 6**
Loading and settlement with time



# Design and construction of the Lukasrand Microwave Tower, Pretoria

Cliff McMillan

**Architects:**
**South Africa Public Works Department, Architecture Department**

### Introduction

In 1973 the Public Works Department appointed Ove Arup and Partners as consulting civil and structural engineers for the design, documentation and supervision of a new microwave tower at Lukasrand, Pretoria. Initial design meetings were held immediately to obtain the brief from the user client, the Department of Posts & Telegraphs. This included the following main requirements:

(a) As an important link in South Africa's microwave telecommunications system, the tower mainly serves a technical function. Functional requirements include the provision of three antenna platforms at the top of the concrete portion of the tower, with equipment and technical rooms immediately below them and a steel mast on top.

(b) Because of the dominant location of the tower in relation to Pretoria's skyline and the numerous important buildings in the vicinity, particularly the UNISA campus nearby on the same ridge, the tower had to be an architectural statement of quality appropriate for the capital city.

(c) There was to be public access to a viewing platform but this should not interfere with the technical functioning of the tower. Restaurant facilities were not required.

(d) There was to be a two-level building serving the tower at the base, with related roads, parking, stormwater drains and other civil engineering services.

(e) The height of the tower was limited by considerations relating to the flight path to Waterkloof Airport.

Work commenced immediately on the design, but in 1975 the project was delayed for nearly two years due to lack of available funds and the general economic climate prevailing. Work was however resumed in March 1977, enabling construction of the tower to start towards the end of 1977 with a target completion date of the end of 1979.

### Design concept

The design concept evolved through very close collaboration between the PWD Architectural Department and the consulting engineers. Six different alternatives providing the space requirements called for in the brief, were finally developed and checked for structural stability and cost. Small-scale models were made so that their aesthetic merits could be assessed (Fig. 1).

These were:

(a) A circular cylindrical tower with a circular multi-level turret on top

(b) A rectangular shaft with rectangular platforms

(c) A cruciform-shaped shaft with octagonal platforms

(d) Four separate circular shafts, one in the middle and three spaced at 120° around the perimeter, with circular platforms

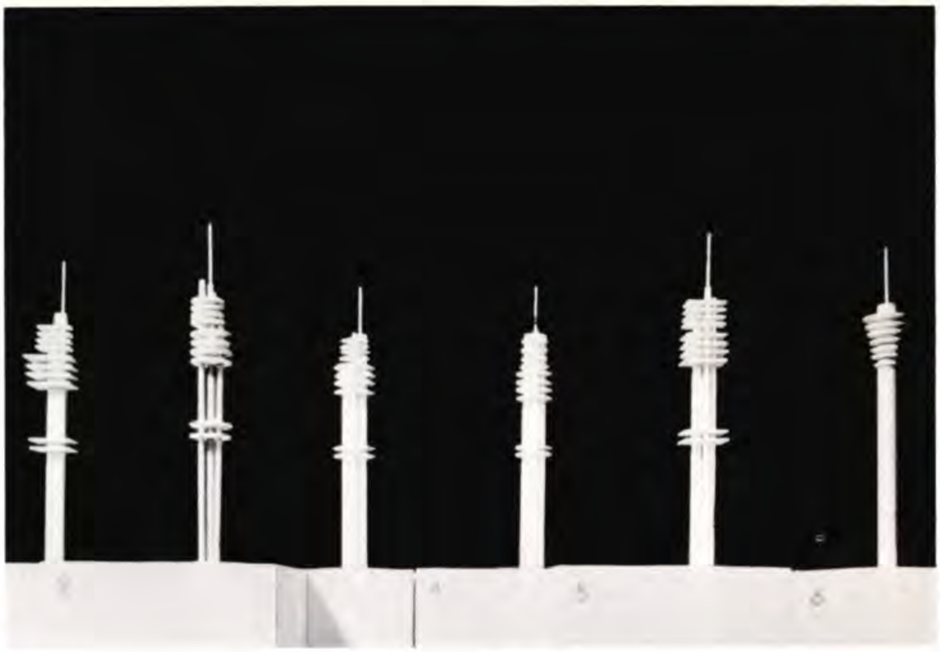
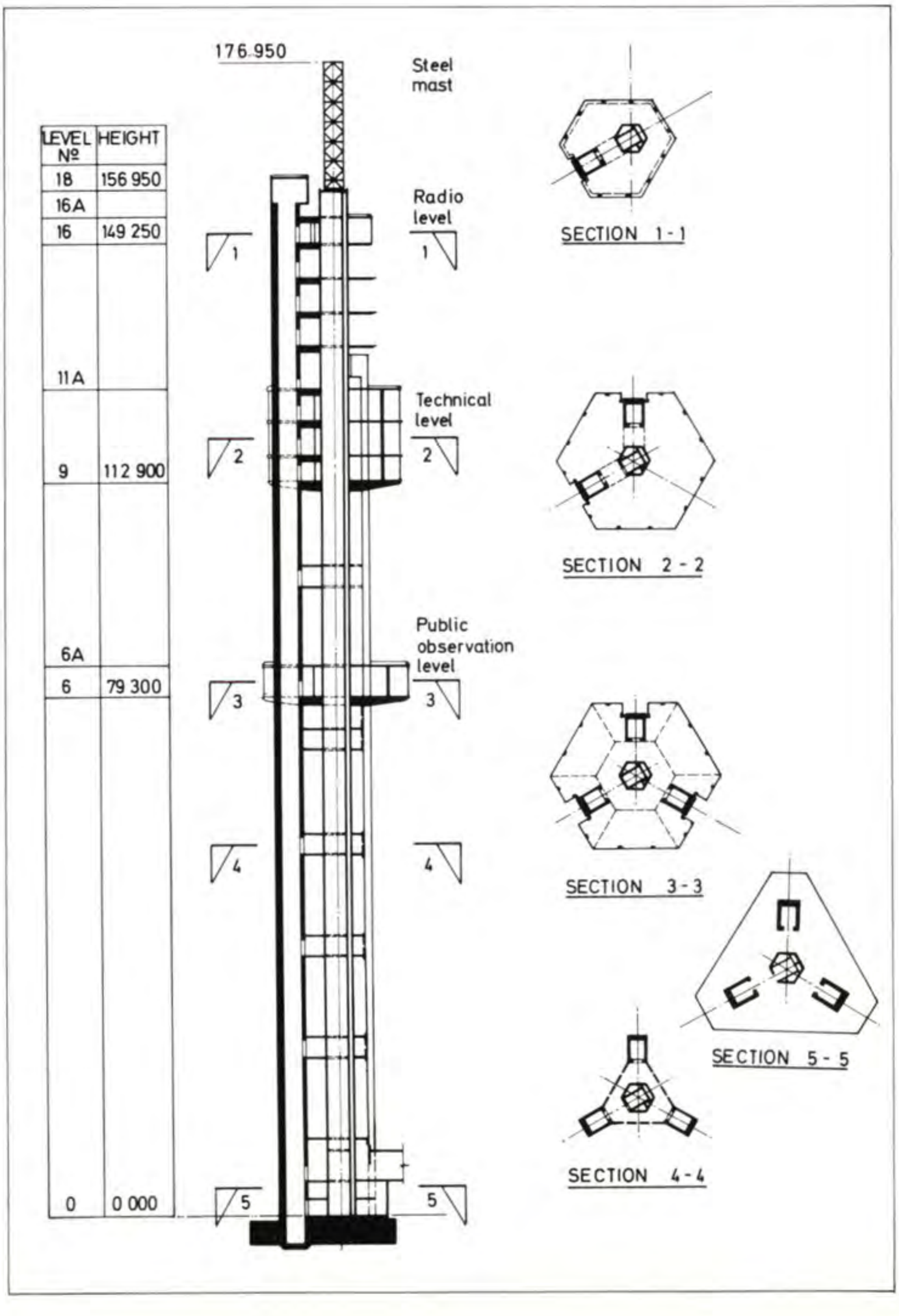


**Fig. 1 ▲**
Alternative forms considered

**Fig. 2 ▼**
General arrangement plan and elevation defining levels and key dimensions



(e) A configuration comprising four separate rectangular shafts arranged in a cruciform pattern with rectangular platforms

(f) A three-cornered 'star'-shaped shaft

(g) A configuration comprising a central hexagonal shaft and three rectangular shafts located at the apexes of an equilateral triangle, with hexagonal platforms.

The simplest and cheapest solution from a structural and construction viewpoint was the circular shaft with circular platforms. There were valid aesthetic reservations about this solution for a number of reasons. The tower is not very tall, being about 157 m to the top of the concrete portion and 187 m to the top of the steel mast. The functional space requirements for lifts, staircase and ducts dictate the size of shaft, making it relatively large in relation to its height and thereby making it impossible to achieve the visual appearance of a tall slender structure. More important, the form did not relate well to the surrounding buildings or topography and would not have created a distinctive architectural statement for Pretoria.

The notion of separating the four shafts arose from the functional requirements. Each shaft serves a separate function, namely three separate lift shafts plus the central core to accommodate the staircase and service ducts for cables, ventilation, plumbing and other services. Moreover the brief called for separation between the access to the public spaces and the technical areas. At the same time it was decided that the public viewing platform could be accommodated well below the technical platforms on the grounds that this would provide an adequate view over Pretoria.

As a result of these factors it became feasible to stop off each of the lift shafts just above the level served, thereby giving a more distinctive unsymmetrical form to the tower. After due consideration of all the factors involved, alternative (g) was chosen. It was decided to terminate each lift shaft just above the highest level it served, namely the public viewing platform, the technical service floors and the top respectively. The result is a distinctive sculptural form which reflects the functional requirements of its various elements, and is in harmony with the predominantly linear appearance of the nearby UNISA buildings.

For structural reasons the three outer shafts and the central core had to be inter-linked at various levels to ensure shear transference between the various vertical elements in resisting the horizontal wind loads and eccentric vertical loads. This inter-connection is provided by a series of wall-beams and platforms at various levels which are integrated into the functional requirements and aesthetics of the tower.

Major horizontal cantilever platforms are required at levels 6 and 9 (Fig. 2). Level 6 supports the public viewing platform and is a tapering hexagonal slab which supports the columns from the roof slab above on its perimeter. Level 9 is a more complex structure, being supported unsymmetrically on the central core and the two remaining lift shafts at that level and cantilevering some 8 m beyond the central core. It carries the three floors and roof of the technical service building on columns supported on its perimeter. It is a concrete slab with a maximum depth of 1.5 m tapering to 450 mm at the perimeter.

### Structural analysis

Because of the unsymmetrical form of the structure and the importance of the interaction between the vertical elements through the wall beams and platforms in resisting wind and eccentric vertical loads, a space frame analysis was necessary. The frame, (Fig. 3), had 186 members and 84 joints and was analyzed for various load cases, including many different wind directions, using our in-house GA18/30 computer. The frame analysis was also used during the conceptual design stage for assessing the required stiffness and strength of wall beams and for determining the natural frequency of vibration for the wind tests using the Rayleigh-Reitz method.

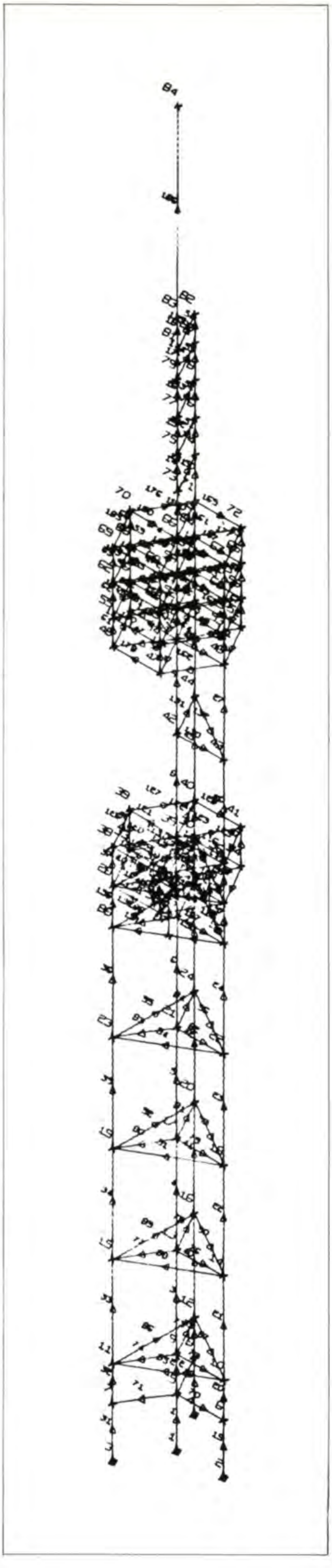
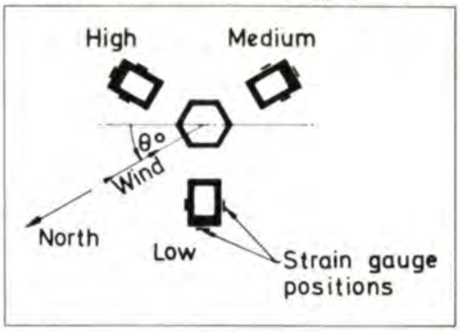


**Fig. 3** ◄
Space frame configuration

**Fig. 4** ▼
Wind orientations and strain gauge positions




CP110 was used as the basis for design and load combinations.

The main cantilever platforms were analyzed elastically using quadrilateral finite elements with the program OVEFINE[1] to determine the magnitude and directions of the principal moments and to obtain the desirable arrangement of reinforcement. The chosen reinforcement arrangement was then checked by yield line analysis.

### Wind loading

Because of the tower's unusual shape, information on wind loading co-efficients and structural response to wind could not be obtained from available sources. It was therefore decided to carry out wind tunnel tests on an aeroelastic model of the structure so that the force co-efficients as well as the aerodynamic excitation could be investigated for various wind directions. This testing was carried out at the University of Pretoria, under the direction of Mr. V. Chasteau[2].

A model scale of 1 :250 was chosen so that the model would fit satisfactorily into the available wind tunnel cross-section of 1.3 m × 0.9 m. The model was made out of 'jelugtong' wood. All external dimensions were to scale while the thicknesses of the concrete in the shafts were modelled as 1.25 times the actual scaled thicknesses in order to give a model stiffness which would lead to a convenient model natural frequency and test wind speed. Discreet metal masses were introduced to model the mass distribution of the tower correctly.

The full-scale natural frequency of oscillation of the tower was determined from the computer space frame analysis to be 0.22 Hz and the torsional frequency as 0.39 Hz.

The damping of the model should be the same as for the full-scale structure, and a value of logarithmic decrement of 0.06 was chosen as reasonable for good quality uncracked concrete. The model was instrumented with three strain gauges on each lift shaft at ground level to measure the vertical strains at various points on the shafts. From these strains the response of the structure for the 12 different wind directions could be determined. The positions of the strain gauges are shown in Fig. 4.

The natural frequencies of the model were measured by two independent methods. The bending oscillation frequency was found to vary between 25 Hz and 33 Hz depending on the direction of oscillation. The torsional oscillation frequency was approximately 64 Hz. The logarithmic decrement for bending oscillation was an average of 0.055 for all directions.

The results of the wind tunnel tests were used to assess the dynamic response of the tower to wind as well as to determine the force co-efficients for the various wind directions for use in the design. In addition, pressure measurements were taken to determine local wind pressures on elements of the tower such as the window wall panels at the public viewing platform and technical levels. To obtain the equivalent drag and lift co-efficients from the test measurements was not straightforward as they had to be derived from six independent strain gauge readings for each wind direction. This was achieved by assuming that the wind velocity profile was known and that the force co-efficient, $C_F$, was constant with height. A space frame analysis of the model was carried out with this pressure distribution in the given direction to determine unit strains at those points where measurements were made. By fitting these results the required co-efficients could be obtained. The resulting force co-efficients for the various wind directions are given in Fig. 5. The average value of $C_F$ was found to be 0.82 and this value was used in the design. These results were later verified by direct measurement of the resultant wind force by means of a wind tunnel balance for various wind directions.

These results are also shown in Fig. 5. The centroid of the wind force was found to be at about 79 m above ground level for all wind directions.

For one particular wind direction ($\emptyset = 30°$) noticeable oscillation of the model occurred within a small range of model wind velocity around 13.4 m/sec (approximately 86 km/hr full scale at the height from which the oscillations were found to emanate or 58 km/hr at 10 m reference height).

The amplitude of these oscillations was halved for windspeed 5% higher or lower than this value, and no oscillations were evident for wind directions other than 5° above or below $\emptyset = 30°$. The oscillations were approximately across-wind and had a frequency of 30.3 Hz with an amplitude which varied with time. Their cause was found to be regular vortex shedding from the portion of the tower above the public viewing gallery and below the underside of Level 9. It was found these oscillations could be controlled by the introduction of aerodynamic fins within this portion of the model.

However, in view of the following it was decided that no such precautions were necessary:

(a) The frequency of occurrence of the disturbing wind velocity and direction is less than once in 10 years.

(b) The maximum resulting acceleration on the tower would be 0.82% g, whereas 1% g is considered to be the limit of acceptability and 1.5% g disturbing.

(c) The amplitudes measured were well within the acceptable limits for the structure.

No other oscillations were detected up to a maximum test speed equivalent to a full scale wind of 144 km/hr.

## Foundations

Boreholes drilled at the positions of each shaft indicated a succession of alternating layers of intact and weathered shale down to depths of 30 m. Core recovery in the weathered shale was very poor, and it was estimated that the percentage of weathered material in the upper 20 m below founding level would vary between 20% and 30% for the four shaft positions.

Two alternative foundation solutions were considered, namely a deep shaft under each of the main shafts and a raft foundation. The raft foundation was preferred on the grounds of its lower cost, shorter construction time and reduced construction risk. However there was concern about possible differential settlement and rotation of a raft because of the weathered strata. Calculations based on the percentages of weathered material observed in the boreholes and on the assumed elastic modulus of 400 MPa for the weathered material, indicated a total vertical elastic settlement of the order of 25 mm and negligible consolidation settlement.

Differential settlements arising from this and wind loading proved to be within acceptable limits, the maximum calculated rotation being less than 0.0015 radians (0.09°).

It was desirable to check the elastic modulus of the weathered material as this governs the deformations. Laboratory tests on small samples were not feasible because of the difficulty of sampling the weathered material undisturbed. It was therefore decided to carry out down-the-hole tests using a pressure meter.

Four NX boreholes were drilled to depths of between 26.4 and 29.1 m. They were immediately filled with cement grout, left to set overnight and then re-drilled the following day. The pressure meter was a pneumatically operated instrument with a 75 mm diameter probe which was buried into the borehole.

Tests were carried out in the weathered material only. Although the majority of the tests failed due to breakages of the pressure cells, the results obtained were sufficient to confirm that the assumed elastic modulus was conservative and that the raft solution could be adopted without concern about differential settlement. The results of the tests produced values of elastic modulus between 580 and 7900 MPa.

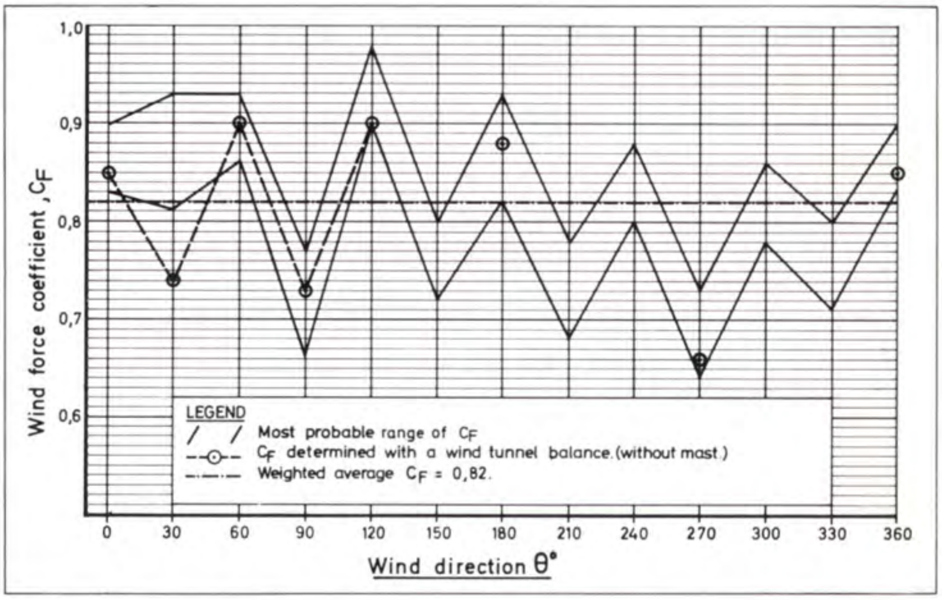



**Fig. 5 ▲**
Variation of wind force co-efficient with wind direction

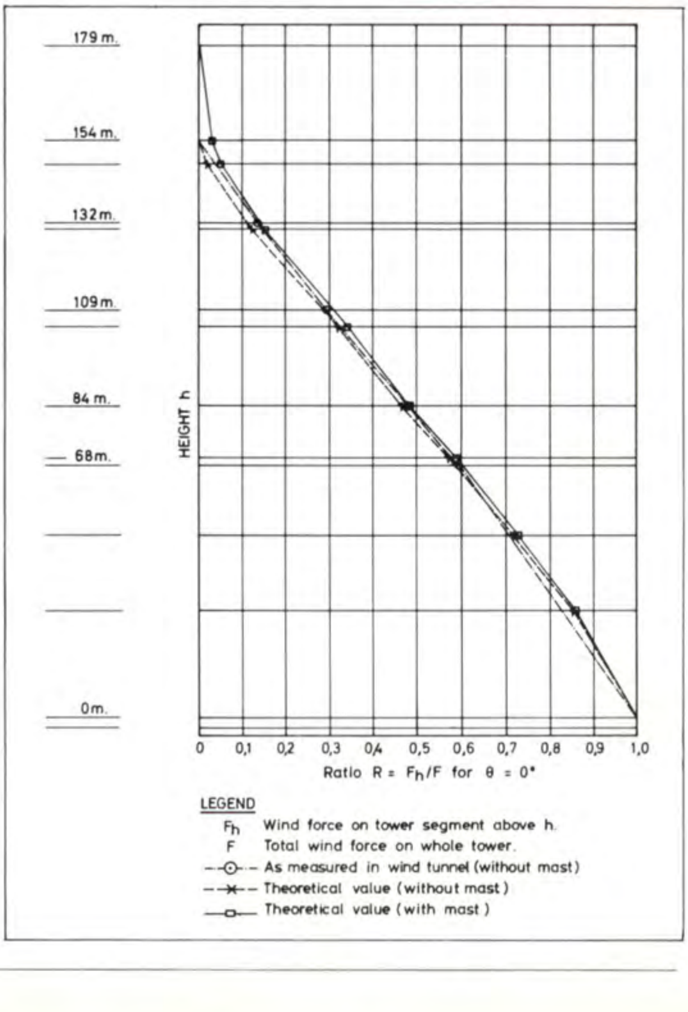



**Fig. 6 ▶**
Distribution of wind force with height

The raft was 3.5 m deep and designed to ensure that the centroid of the dead load of the tower coincided with the centroid of the raft with a maximum bearing pressure of 600 kPa under dead plus live load and 700 kPa with wind loads included.

**Construction**

A number of special construction problems existed, namely:

(a) The construction of the raft foundation.

(b) The method of construction of the vertical elements, their inter-connection with the wall-beams and the control of alignment and verticality.

(c) Control of the quality of concrete finish on the vertical shaft elements and the special off-shutter concrete finish required on the low rise building at the base of the tower[3].

(d) The construction of the major horizontal cantilever floors at Levels 6 and 9.

(e) Detailing and construction of the window walls.

Because of the special nature of the project, the client adopted our recommended procedure of pre-qualification of tenderers. In this way a short-list of invited tenderers, fully qualified to handle the project and briefed on the design intent, was obtained.

**Raft construction**

It was imperative that a high quality concrete finish was obtained. To achieve this, great care was taken with the concrete mix design and the choice of aggregates. This was done with assistance from the Portland Cement Institute. Test panels were manufactured prior to tender using various concrete mixes to examine both the off-shutter finish and the concrete colour. To avoid changes in the colour of aggregates during the construction period, the contractor was asked to stockpile all the aggregate required for the project prior to commencement of construction. Despite this precaution, the sand ran out towards the end of the project and as the supply at that quarry had changed colour since the start of the contract it was necessary to match the sand from elsewhere.

The 3.5 m deep raft involved some 1,400 $m^3$ of concrete which was cast in three separate horizontally-layered pours.

The maximum depth of pour was 1.5 m. Heavy reinforcement only occurred in the bottom and top pours and could be fixed only when required, thereby avoiding the need for excessive support for the top reinforcement. Each section was protected with polythene sheets for curing after casting.

Precautions were taken with the mix design to minimize the heat of hydration by using a 50/50 combination of cement and slag and large 40 mm aggregate to reduce the water demand of the mix. Temperatures were measured by means of thermo-couples through the depth of the raft after casting and differential temperatures remained within acceptable limits.

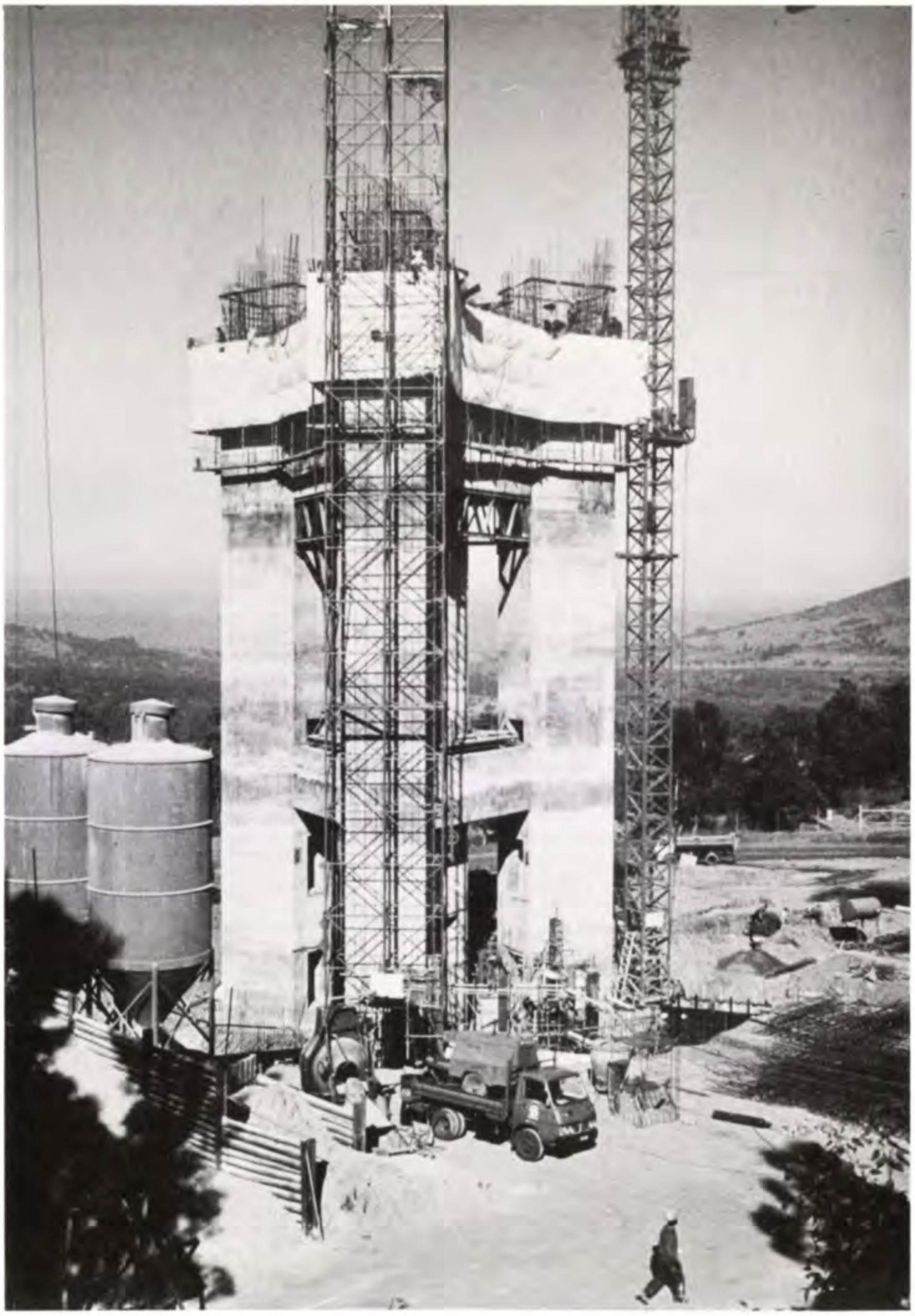


**Fig. 7**
The partially completed tower showing the sliding platform

**Sliding formwork**

The contractor proposed a sliding form of construction for all the vertical elements including the wall-beams. The procedure was to slide continuously to the underside of each wall-beam, at which stage a steel soffit girder was installed between the shafts and fixed by means of welding to plates cast into the walls. These girders formed the soffit of the wall-beams and supported jacking rods to enable the wall-beams to be slid concurrently with the shafts after making minor changes to the shutter (Fig. 7).

To achieve the required high standard of finish on the vertical elements a steel shutter was used with jacking rods positioned to miss the openings in the shafts. (Where jacking rods had to go through openings, temporary columns were cast around the jacking rods to give them the required support). The sliding platform consisted of three levels; namely, the shutter level, finishing level and the curing compound application level, and was braced to ensure rigidity during sliding. The sliding shutter incorporated sections which linked the lift shafts and were used in sliding the wall beams.

Before commencement of the slide, two small test slides were conducted using different concrete mixes, and as a result the most suitable mix satisfying both slidability and finish was chosen.

In early discussions with the contractor it was decided to use reinforcement cages in the shafts tied together with the lighter horizontal reinforcement. The height to which horizontal reinforcement could be fixed was dictated by the jacking frames which span the shutter. The cages were staggered vertically to enable the reinforcement to be placed in a continuous sequence thereby avoiding a bottleneck in the fixing operations.

During the sliding of the deep wall-beams it was more difficult to control the surface finish. The heavy wall-beam reinforcement was placed prior to sliding, it being possible to fix a maximum height of only about 900 mm at a time due to the interference of the jacking frames. This portion was then slid before fixing on the next could proceed. During this stop-start procedure for sliding the wall-beams, and in particular the 8 m deep beam between levels 5 and 6 which took a total of 225 hours, care had to be taken with the concrete preparation at each stop to avoid blemishes at the construction joints.

Where necessary, screw couplers were used to connect the main reinforcement in the slid concrete portion to the beams and slabs to be constructed later. This again meant a slowing down of the sliding speed and in some cases a stop, and as a result special care was necessary with the finishing.

The alignment of the slide was controlled by means of optical plummets set in the shaft corners at ground level. Readings were taken on a target on the underside of the sliding platform. External checks were carried out using a single-second theodolite set up over beacons near the base of the tower. In the later stages a laser beam was also used, but this did not prove as successful as the optical plummets because of the divergence, shape and movement of the beam. Also, the shape of the tower did not lend itself to the use of a single laser. Because of the rate of sliding (up to 200 mm per hour) the optical plummet readings had to be taken every few hours.

Each jacking frame had a water level fed by a central reservoir for levelling the shutter. By sliding with the shutter slightly off level the alignment could be adjusted. A lever device was incorporated into the sliding shutter to enable a force to be applied to adjust the alignment of any shaft.

Difficulties were experienced both in obtaining consistently reliable readings with the optical plummet and with maintaining alignment, particularly because the balance and response of the sliding shutter appeared to be upset when successive shafts stopped off, resulting in a different shutter configuration. Generally the alignment was controlled to within 50 mm, but at one stage a deviation of nearly 100 mm was detected on the northern shaft. This was gradually corrected and is not visible and does not impair the performance of the lift.

Finishing of the concrete surface was done on the second level of the sliding platform. Under normal sliding conditions the concrete emerged from the underside of the sliding forms relatively 'soft' (six hours old). The surface was then rubbed down using a wooden float and brushed vertically to give it an even appearance. After a stop in the sliding, the first concrete to emerge from the forms was hard, and to get an equivalent finish the surface was first rubbed with carborundum stones and then lightly brushed with a grout made by sieving the coarse aggregate out of the concrete used in the slide.

Due to the difficulties involved in getting uniform distribution of water for curing, and the possibility of freezing during the winter months when the majority of sliding took place, water spray curing was not used. Instead, a resin-based curing compound (*Cormix CM 90*) was sprayed onto the concrete by hand from the lowest level of the sliding platform. During the early stages of sliding, the spray equipment was not adequate to ensure even distribution of the compound and over-application occurred in certain areas, leaving yellow marks. These have almost entirely disappeared with time.

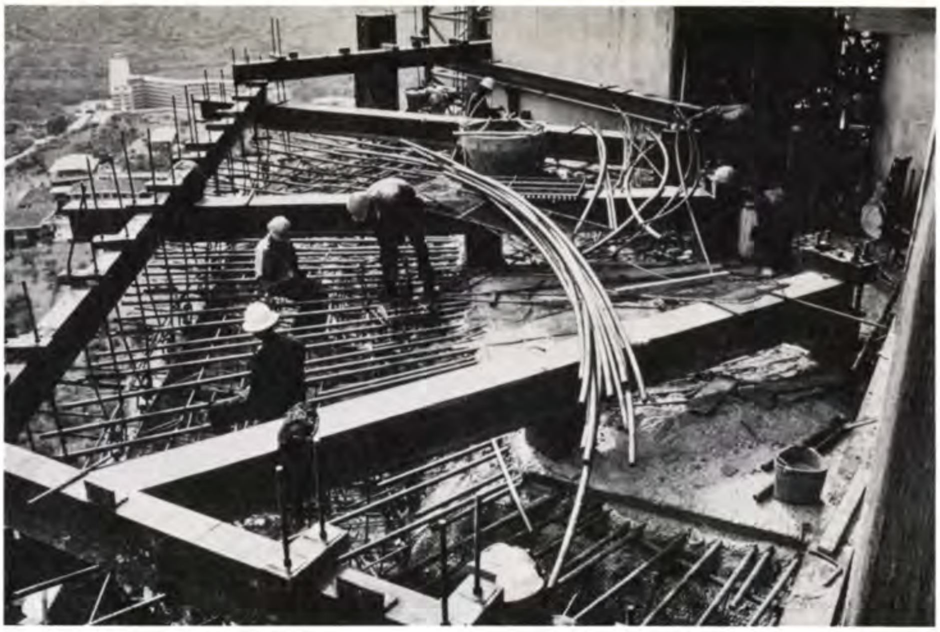


**Fig. 8 ▲**
The sliding platform showing girders, hangers, precast slabs and reinforcement

**Fig. 9 ▶**
Ground level off-shutter concrete

### Cantilever platforms

Detailed discussions were held with the contractor on alternative proposals for the construction of the main cantilever platforms at levels 6 and 9 and we were appointed to design the temporary works on behalf of the contractor.

The final solution adopted was a launching system using precast concrete panels approximately 1.5 m square and 100 mm thick as permanent soffit shutter panels. The pattern of joints between panels was detailed to meet the architects' requirements. The slab at Level 9 is technically the most difficult, being supported unsymmetrically by the central core and the two remaining lift shafts at that level and cantilevering some 8 m beyond the central core. (See Fig. 2 section 2-2). It tapers from 1.5 m deep at the core to 450 mm at the edge where it supports the columns carrying the technical floors and roof above.

A concrete annulus about 1.5 m deep was first cast around the shafts using conventional shuttering supported on steel brackets off the vertical shafts. Radial structural steel cantilever beams were then attached above this concrete and supported a series of transverse beams at their extremities. The precast soffit panels were supported on the completed concrete on the inside and hung by adjustable hangers, from the transverse beams on their extreme edge. The joints between panels were sealed using a sealing cord and grout. The reinforcement was fixed also using the steel cantilever system for support and the stop-ends were positioned before casting the concrete. The stop-ends were located so as to allow the hangers to be re-used.

Once the concrete had reached the required strength, the hangers were loosened and the whole system moved out using the new concrete for support. The next row of precast panels were then hung and the process repeated.

To finish the edge strip, steel shutters were supported from the cantilever system. The same system was used for both levels 6 and 9, there being four rows of panels and the edge strip at Level 6 and two rows of panels and the edge strip at Level 9. The initial in situ annulus was more difficult to construct at Level 9 because of the absence of a shaft on one side. This was replaced by a steel cantilever bracket in that position.

Due to the critical nature of the hanging details, a full-scale load test was carried out on one of these precast panels at the SABS. The girders, hangers, precast slabs and reinforcement are shown in Fig. 8.

An interesting facet was the detail developed for the stop-end between pours. This consisted of a lightweight frame covered with expanded metal, which enabled the heavy cantilever reinforcement to be placed simply by cutting holes through it where required. A certain amount of shutter oil was left on the precast panels and some grout had leaked from the joints. It was therefore necessary to clean the underside of the slabs after construction, using a lightweight structural steel platform hung under the slab. After this treatment a very satisfactory soffit finish was achieved.

### Off-shutter concrete on the low-rise building

The finish required for the low-rise building at ground level was to express the grain pattern of sawn timber boards on the surface. To achieve this, tongued-and-grooved boards were sandblasted to enhance the grain pattern and then sealed. Joints were sealed by means of compriband strips against the timber mock-mason joints.

The design mix for the off-shutter concrete was developed in consultation with PCI. Strict site controls were set up to monitor the stockpiling and use of the various mix constituents.

The details of the formwork for each panel were drawn up by the contractor and submitted for approval. To gain as many re-uses from each form as possible, the wooden mock-mason joints were made movable on the form. This was achieved by the use of compressible sealing strips to seal it against the 'rough' timber grain. This meant that, within the restriction of the board pattern, one form could be used for any size of panel.

All construction joints had to fall on mock-mason joints and to prevent grout leakage the forms were sealed onto existing concrete using a compressible sealing strip on the edge of the completed concrete. All details of ferrule holes, expressed joints, colour of concrete, concrete finish, etc. were resolved between the architect and the contractor prior to starting construction. (Fig. 9).

To ensure that the required quality of finish would be achieved, a number of sample panels were prepared prior to commencing construction and compared with a test panel which had been produced prior to tendering and was specified as representing the required quality.

**Fig. 10**
Tower during construction

An interesting aspect of the shuttering was the method used for construction of openings for windows, etc. To avoid the problem of concrete not reaching the centre of the base of an opening, small temporary columns were constructed within the opening. This allowed placing and vibration of concrete in the inaccessible areas.

Forms were generally stripped in under 24 hours which meant that if some patching became necessary, it could be undertaken with minimal effect on the final finish. These areas were patched using grout and a sponge.

Water curing was used, the walls being sprayed with water twice daily. A total of 2,000 m$^2$ of off-shutter concrete was cast in the ground level structure.

### Window walls

The window walls which occur between Levels 6a and 6, 9 and 11a and 16 and 16a. consist of a series of structural steel mullions hung from the upper levels and supported horizontally at each level, into which the window frame is fixed so that all glazing is done from the inside of the building. An additional feature is the use of single glazing and external walkways, enabling the glazing to be cleaned from the outside as well as the inside of the building and thereby eliminating expensive pivoting window frames.

Special care was taken with the specification for this steelwork.

A mock-up was made to study the details and test the system against rain penetration. As a result of the tests, modifications had to be made to meet the architectural requirements and still ensure adequate corrosion protection. The specification adopted was 150 microns of bronze metal spray over 100 microns of zinc metal spray, sealed with a clear acrylic sealer.

### Credits:

*Architects:*
South Africa Public Works Department, Architectural Department

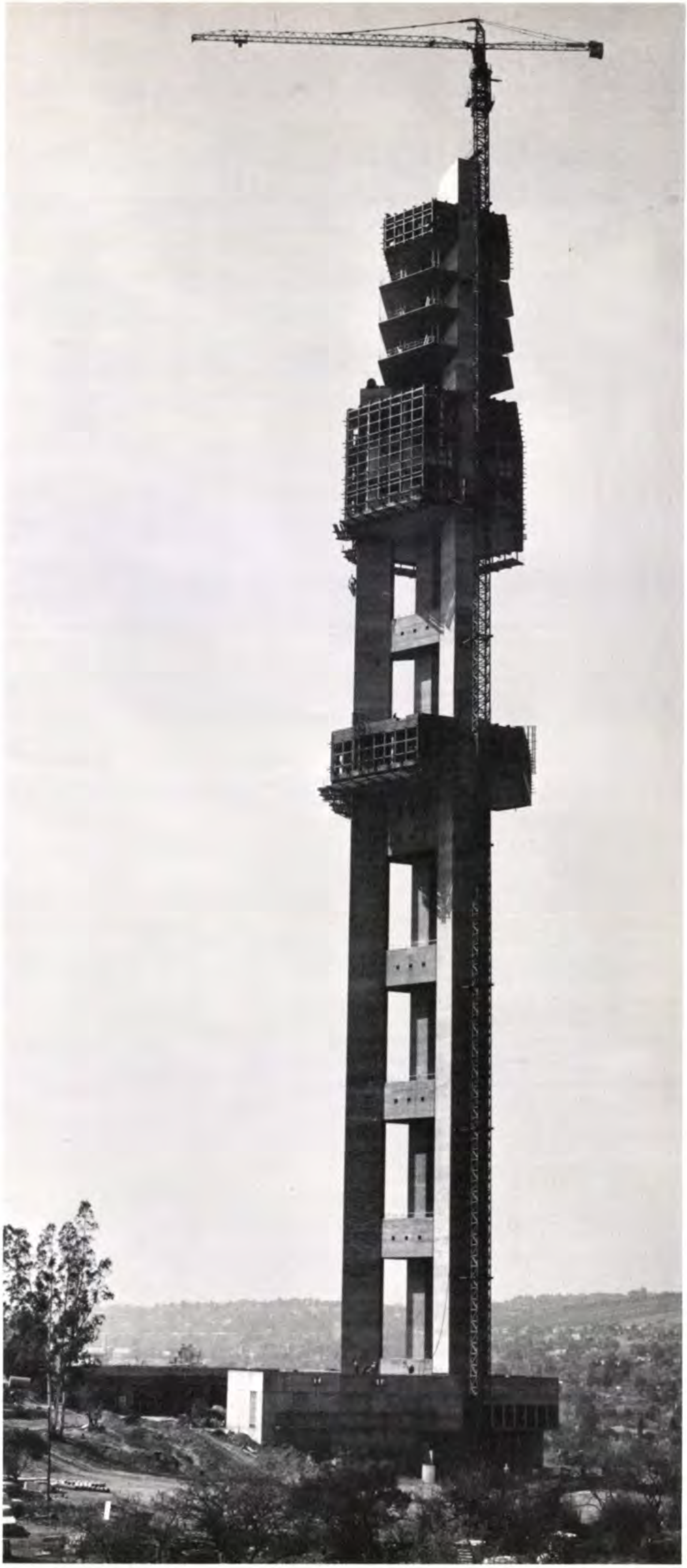
*Quantity surveyors:*
Southby, Bihl, Detert & Slade

*Consulting electrical engineers:*
Bidermann, Finn, Beekhuizen & Partners

*Mechanical services:*
South Africa Public Works Department, Mechanical Department

*Main contractors:*
Stocks-Futurus (Pty) Ltd. 2 – a consortium between Stocks and Stocks (Pty) Ltd. and Futurus.

The author is indebted to the many individuals who formed part of the team which made this project a success: firstly, to the relationship which existed throughout between the architect responsible, Mrs H. Lenddorff, the consulting engineers, the PWD engineers and the quantity surveyors, out of which it was possible to create a tower of quality; secondly, to those members of Ove Arup & Partners who participated tirelessly on the project, particularly Jan Heynike for his contribution to the development of the basic concept and to the engineers who executed it, Vincent Diesel, Ron Finkelstein, H. Scott and the late Fred Hoogendijk; thirdly, to Mr V. Chasteau for his contribution in advising on and carrying out the wind tunnel testing; finally to the contractors, and particularly Dr. W. Vance, for the excellent relationship which prevailed on site and the enthusiastic manner in which challenging problems were met and overcome.



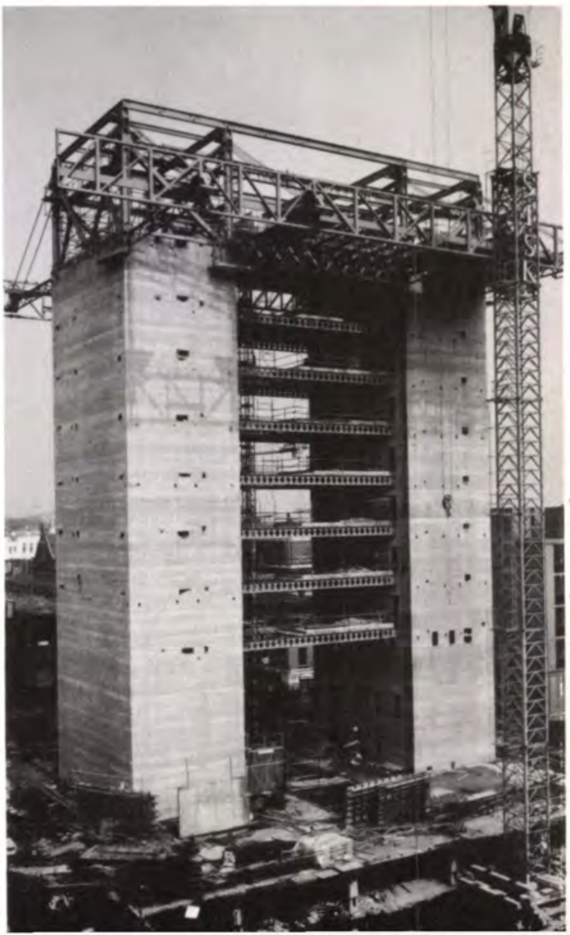
# Central Bank, Dublin

**Architects: Stephenson & Associates**

The Association of Consulting Engineers of Ireland has given its 1980 Excellence Award for a structural project to the Central Bank, Dame St. Frank Lydon of Ove Arup & Partners Ireland, consulting engineers for the project, accepted the Award at the annual dinner of the Association on 7 March.

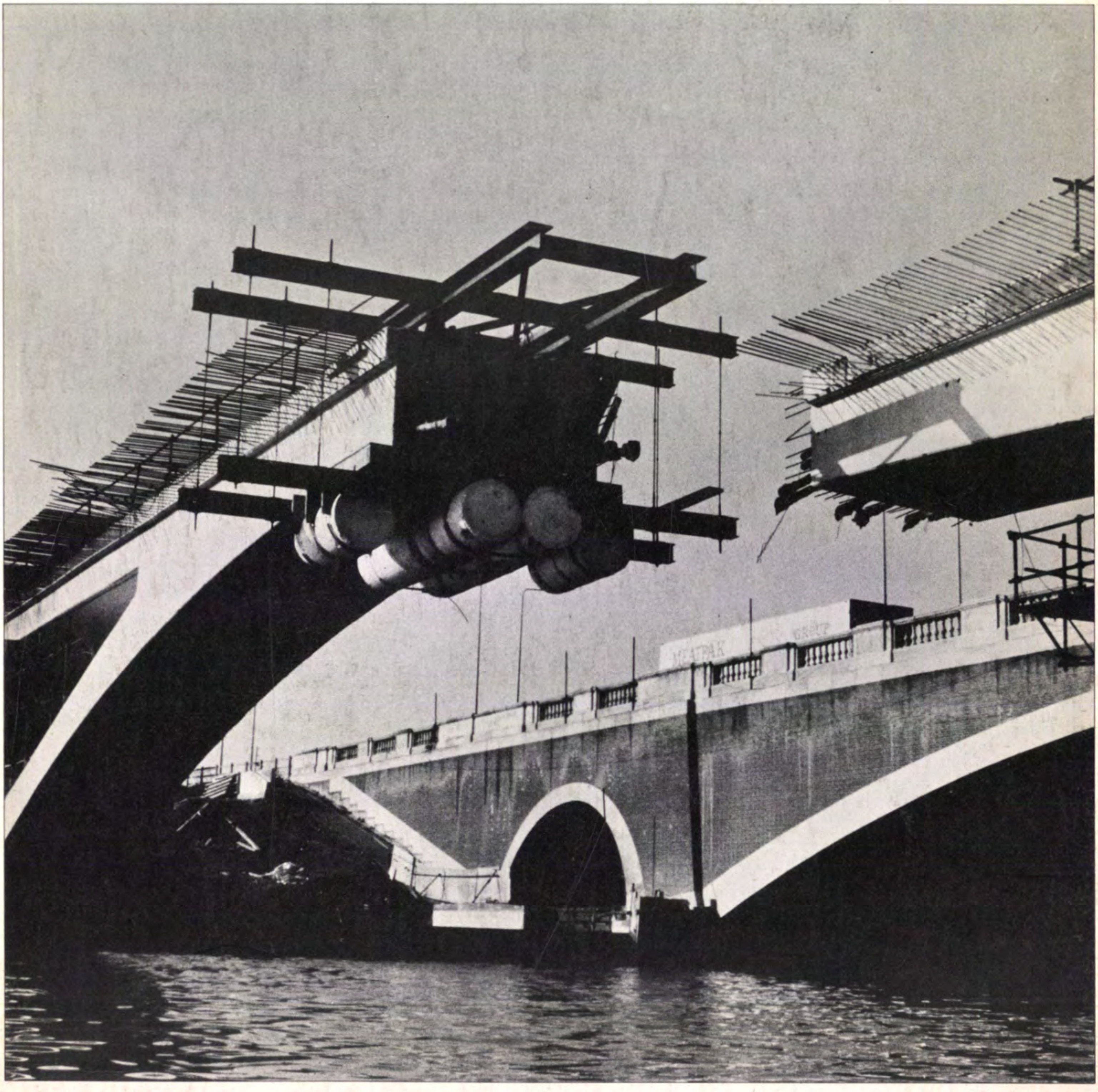
The main building is an eight storey, 45m high structure and measures 45m × 45m on plan. It is set towards the north of the site overlooking a landscaped piazza fronting onto Dame Street. Two ancillary buildings are also incorporated on the site. One of these accommodates the staff dining facilities. The other is a reconstruction of a listed 19th century building which stood on the site prior to this development. There is a two storey underground car park covering most of the site to accommodate 130 cars.

The principal feature of the building design of engineering interest is the superstructure which is suspended from the top of two reinforced concrete service cores. Other aspects of the building construction, the use of slip forming to construct the concrete service cores and the lifting into position of complete floors assembled at ground level, are also unusual features.









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