THE ARUP JOURNAL

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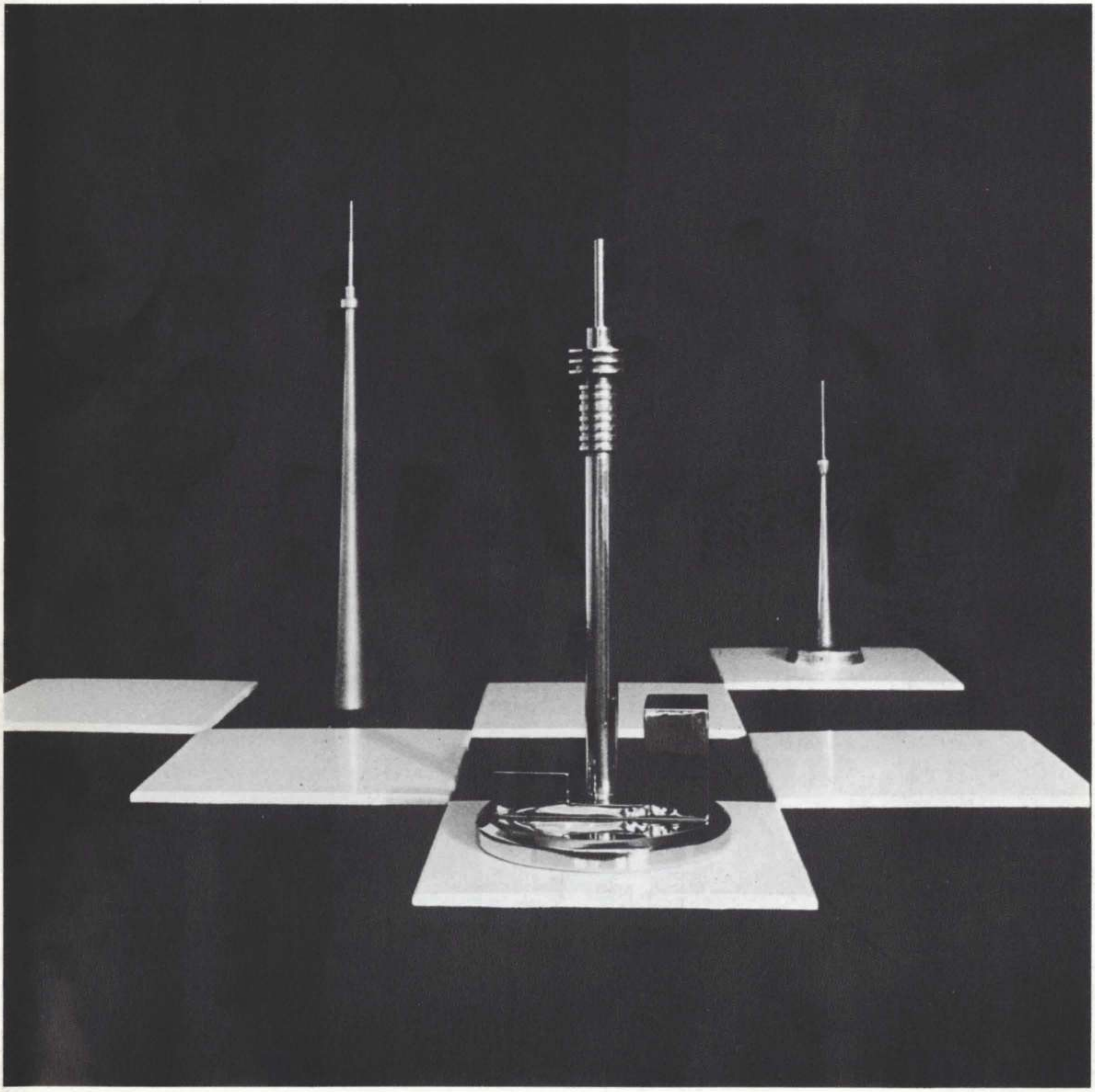

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Contents

Front cover: Models of Hillbrow Tower (foreground) Albert Hertzog (rear right) and Emley Moor Tower (rear left) (Photo: Harry Sowden)

Back cover: Jan Smuts Airport waiting area (Photo: Lynda Shave)

Investigations of structural failures

Poul Beckmann

This article is based on a talk given to the Nottingham Society of Engineers on 27 November 1972.

My interest in investigating failures is aroused firstly through my curiosity to find the real reason why, secondly by the challenge of finding a way of putting it right, and thirdly by the part of my brief which is to prevent somebody in the firm repeating the mistake.

I shall in the following be stating personal opinions arising out of my own experiences. If I should appear to lay down rules or state principles of universal applicability, it just so happens that these are my strong personal convictions.

I am not enthusiastic about investigations which are aimed only at apportioning the blame for the purpose of impending lawsuits. This activity is generally unconstructive and tempts the investigator to adopt a nit-picking attitude in order to heap up evidence against the opposing party. The general climate surrounding the building industry today is such that there is a large demand for this type of investigation. This is unfortunate because it may lead to an attitude of mind in which investigators forget that very few contracts go strictly according to the textbook all the time, and that indeed, if everybody stuck rigidly to the rules, hardly any buildings would (ever) be finished anywhere near on time.

For the purpose of our particular subject, and for that purpose only, I will include under 'failure': collapse, excessive deformation, cracking and any other kind of severe structural misbehaviour because otherwise I shall be short of examples. However I must emphasize that I do not accept this definition when it comes to general assessment of design; there I distinguish sharply between 'collapse' and 'local damage'.

Gathering evidence with an open mind

Having started my career by designing building structures, and having lived through a number of fair-sized contracts, I try at the start of every new investigation to tell myself firmly 'There but for the grace of God go I', just in case I should be tempted to adopt an attitude of academic arrogance towards the original designer or the contractor.

The next thing I try to do is to go and 'view the body' as soon as possible before anybody starts tidying up, and in the process, removes some piece of indirect evidence. By this, I mean that the actual juxtaposition and/or stratification of the debris, if any, can hold vital clues. For instance, after the Tay Bridge disaster, the main girders were found only 10 m from the feet of the piers whereas the actual pier trestles were 30 m high. This meant that the piers could not have toppled intact by being blown over bodily. This in turn meant that the disaster was not caused solely by failure to bolt down the cast iron piers against overturning from wind.

The impressions of a first site inspection tend to be chaotic and it is difficult to see at first what is crucial and what is not, and it is even more difficult to remember the shape and juxtaposition of the various bits and pieces. There is nearly always some urgency in clearing up the mess and for this reason, I think it is as well on one's first visit to make a fairly extensive photographic record of what one has seen. This will enable one to study, in the relative calm of one's office, the visual evidence, and compare it with the drawings and calculations of the original design, which usually take a little while to obtain, and which, on their own, may give scant guidance on what to look for.

A study of the principles of the design of the structure, and in particular the detail design (as opposed to the numerical analysis), is the necessary next step, and after that one will often go back for a second look at specific details of the failed structure or perhaps for a confirmation of verbal evidence from people who may have witnessed what happened. (It is worth remembering in this connection that 'responsible' people usually will be protecting some interest or other, and that for this reason, their verbal evidence may not be all that helpful in clarifying what really happened!).

The necessity of formulating hypotheses

Up to this point, it is as well to keep an open mind on the cause of the trouble because only by doing so will one give equal coverage to all the available evidence, and failure to do so can mean that some essential fact does not get recorded in time, may be tidied away, and thus not be available when it becomes apparent that one's first hypothesis was wrong. I believe however that, before one starts a detailed analysis of anything in particular, one should formulate one or two hypotheses on which to base the further studies.

The ease with which computers can process data has led some research workers to adopt a policy of amassing all the available facts they can lay their hands on without attempting any discrimination, in the hope that the computer will sort it out for them eventually but, in the case of failures, some bits of the jig-saw puzzle which the investigator is trying to assemble may be lost forever and their shape can only be guessed by the holes which are left when all the other bits have been put together.

Computers are not very good at guessing, nor at working from incomplete data, and for this reason, I would much rather propose one or two hypotheses for the cause of a failure and proceed to prove or disprove them with the available evidence.

The need to look beyond the Codes of Practice

At this stage, when one is trying to analyse what has happened, one must clear one's mind once and for all of the notion that things fall down because a conventional calculation shows stresses to be somewhat higher than those allowed by the code of practice.

Codes of practice are generally formulated so that conforming designs provide not only agreed factors of safety against failure, but also acceptable service performance of structures, and they have until recently generally assumed fairly summary design procedures involving only traditional, approximate, methods of analysis. Rarely do they take full account of the real behaviour of structural elements because this is too difficult to calculate in the limited time and for the limited money which is available for the design of most one-off building structures. (It should be remembered that while a single design of a car will usually be built by the thousand and be modified in the light of experience, the designer of most building structures and bridges is given only one attempt in which to get it right. Furthermore, the man who meekly puts up with having his car called in by the manufacturer for replacement of a vital part from a faulty batch, will take his contractor, his architect and his consulting engineer to law at the slightest suggestion that he has not had a Rolls-Royce quality building for the price of a Ford).

But I am digressing; it is irrelevant whether or not the stresses are 10% over the 'permissible', because the stresses in the codes usually allow much wider margins. What does matter is whether the assumptions for the design are reasonable and have foreseen conditions which, although possible and indeed probable, may not have been included in the design brief and/or the codes.

It has in my opinion been established repeatedly that compliance with codes of practice does not always prevent failures, and one can make a case that the more elaborate the codes of practice become, the more likely we are to get failures because designers will spend their mental energy making sure that the design complies with the code instead of thinking about whether or not the structure they are designing will be sound. I think that it is generally this lack of appreciation of real structural behaviour and lack of imaginative forethought, which can be seen in the end to have been the cause of most failures where the design was proved to be at fault.

For the Ferrybridge cooling towers, the design brief specified wind loading which we to-day consider inadequate, but added to this there was a failure to envisage what would happen to the stresses in the walls if the wind speed should exceed the amount specified and, as stresses in this case increase at a far greater rate than the wind loads, trouble was never far away, despite the fact that the codes of practice were followed.

The original design for the Tacoma Narrows bridge catered for drag forces from quite high wind velocities. It failed however to take account of vortex liberation from alternate edges during moderate winds and of the effects of resonance between vortex formation and the natural frequency of the torsional oscillations of the bridge.

In both these cases, the 'traditional' design criteria were satisfied, no doubt with a lot of back-up calculations, but nobody had asked 'What happens if . . . ' In other cases, the question may have been asked, but the wrong answer given because the 'State of the Art' had not progressed enough. If in such a situation the designer does not recognize his ignorance and makes provisions over and above the official requirements, he may invite serious trouble although he had not been negligent.

Naïve design is however by no means the root of all troubles. Mistakes on site and bad workmanship can play havoc with a perfectly good design and can in some cases deprive an indifferent design of the margin of safety which might otherwise have ensured its apparent success.

And then there are situations when things ought to have fallen down due to gross abuse or mis-use.

With all these possibilities to choose from, how does one proceed to propose hypotheses for the causes of a particular failure? My experience does not cover enough cases to enable me to offer any generally applicable methodology, but perhaps the following examples of what was done in particular cases may give some food for thought about the generalities.

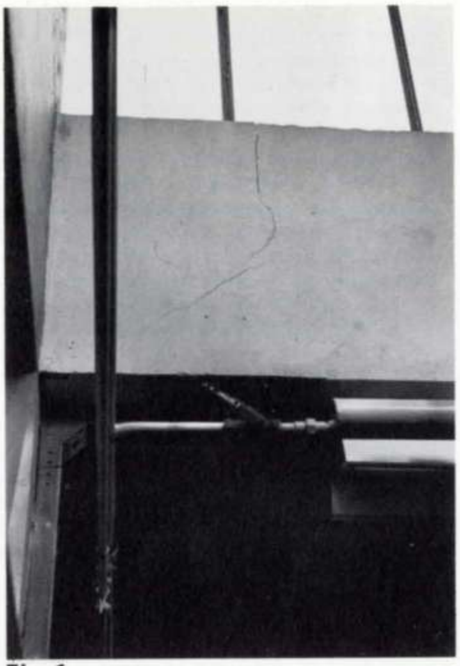


Fig. 1
Crack in roof beam

Fig. 2
Close up of top of crack in roof beam

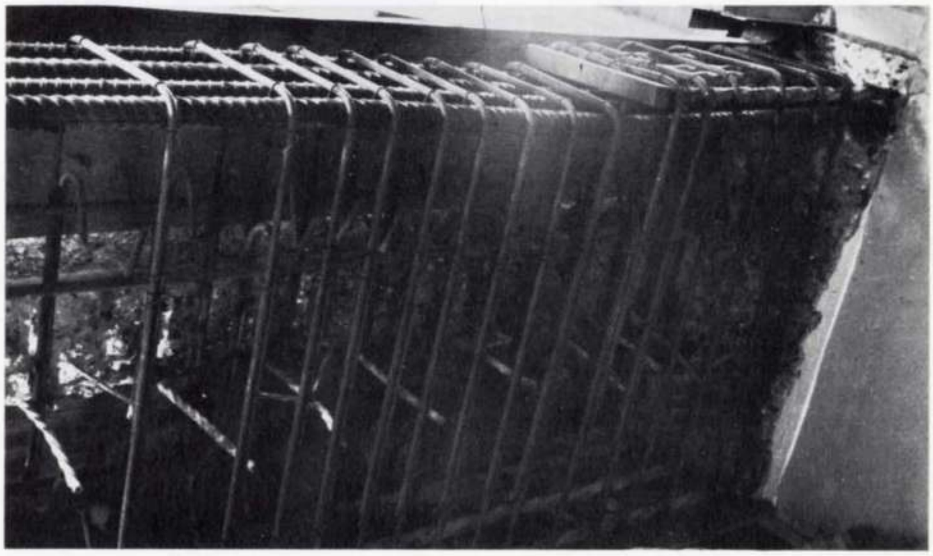


Fig. 3
Remedial work to roof beam

The case of the missing top bars

Some years back, I was asked to look at some cracks which had appeared in the reinforced concrete beams to the monitor roof of a workshop block of a technical college. The beams were of rectangular section, continuous, spanning 12 m between columns which also supported transverse beams on the line of cross walls. The complaint arose primarily because a piece of plaster dubbing-out had fallen down during the commissioning of the building and shown up a rather big crack in one of the beams, whereas in the others they were of the magnitude of normal drying shrinkage cracks, and therefore had not caused undue alarm.

The cracks started from the top of the beams about 0.75 m from the supports and extended nearly vertically down for about half the depth of the beam before curving towards the supports until they became almost horizontal at the level of the bottom reinforcement next to the supports.

The designer had been anxiously checking his calculations but they were soon found to be in order and speculation was rife as to the cause of the cracks.

Having secured the ladder to the part of the beam on the support side of the crack, I started to probe with a cold chisel and found that whereas the drawings required four 32 mm bars over the supports, there appeared to be only two 12 mm bars across the crack in the whole width of the beam.

Further probing showed that the four 32 mm bars had in fact been provided, but had been misplaced so as to stop exactly where the crack started instead of extending the appropriate distance from the support. What had happened was that a bending crack had formed at the end of these bars. As, according to *CP114*, the shear resistance of the concrete section was ample, only nominal links had been provided, and the shear resistance of the cracked section became inadequate, so that a shear crack developed at the root of the bending crack and the bottom bars started to 'delaminate' and all that prevented total collapse was aggregate interlock!

Having ascertained the cause it remained to devise a repair which could be effected without complete demolition, as the building was now generally in use. Approximately 3.5 m of concrete was cut out, new 32 mm bars were spliced on to the protruding 'stumps' and, as the splice length was a little on the short side, cross bars were welded to the ends of the stumps and to the ends of the new bars. Extra stirrups were provided and the cut out portion of the beam was re-concreted. In order to make the repair fully effective in re-establishing the design support moments and thus reduce deflections in the adjacent spans, these had been jacked up by means of flat jacks which were deflated after the repair concrete had reached its design strength.

The case of the cracked diaphragm wall

During the excavation of the basement under a tall block in the City, single horizontal cracks appeared on the insides of some diaphragm walls. At the time, the intended use of the diaphragm walls as part of the permanent works was a fairly novel feature and the depth and sequence of excavation were also outside the traditional experience, so that considerable concern arose from the appearance and subsequent widening of these cracks, and additional reinforcement was provided in vertical chases cut in the walls.

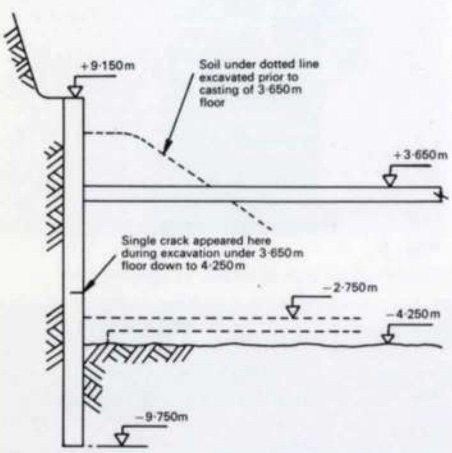


Fig. 4
Construction sequence and crack position in diaphragm basement wall

The phenomenon was however still mystifying and, to check the stresses in the original reinforcement, a bar was exposed adjacent to the crack and had *Demec* studs glued on to it. After reading the *Demec* strain gauge between the studs, the bar was cut outside the gauge length and a second reading gave a release of strain which corresponded to a slight overstress in the bar prior to cutting. This overstress was however not nearly enough to explain the magnitude of the crack and in addition, there was a discrepancy between the measured steel strain and the crack width, the latter suggesting a much more alarming degree of overstress.

This discrepancy between strains prompted one of my colleagues in the firm to the thought that the behaviour of the wall was similar to that of a prestressed beam with unbonded tendons. We began therefore to suspect the bond between the reinforcing bars and the concrete which had been tremied into the slit trench which was filled with bentonite mud, and we instigated a number of tests to prove our hypothesis.

Two model beams to one-fifth full size were made and tested; one beam was cast in a dry mould and the other under bentonite, both by means of a tremie pipe. The test load was applied by small, close spaced, hydraulic jacks, simulating the triangular distribution of soil pressure, and five dial gauges were provided to measure deflections.

It was found that the maximum deflection of the beam cast under bentonite was 60% greater than that of the 'dry' beam and, in addition, the deflected shape of the bentonite beam had an acute 'V' shape whilst the 'dry' beam exhibited the normal rounded sag.

This was a strong indication, but as pull-out tests prior to construction had indicated no substantial reduction in bond of bars dipped in bentonite mud, we felt that further proof was required.

We therefore had a block approximately 0.9 m × 0.9 m cut out of the 0.8 m thick wall by means of thermic lancing so as not to disturb the bond of the embedded bars by vibrations from pneumatic drills. The block was taken to a stonemason's yard in South London where two beam

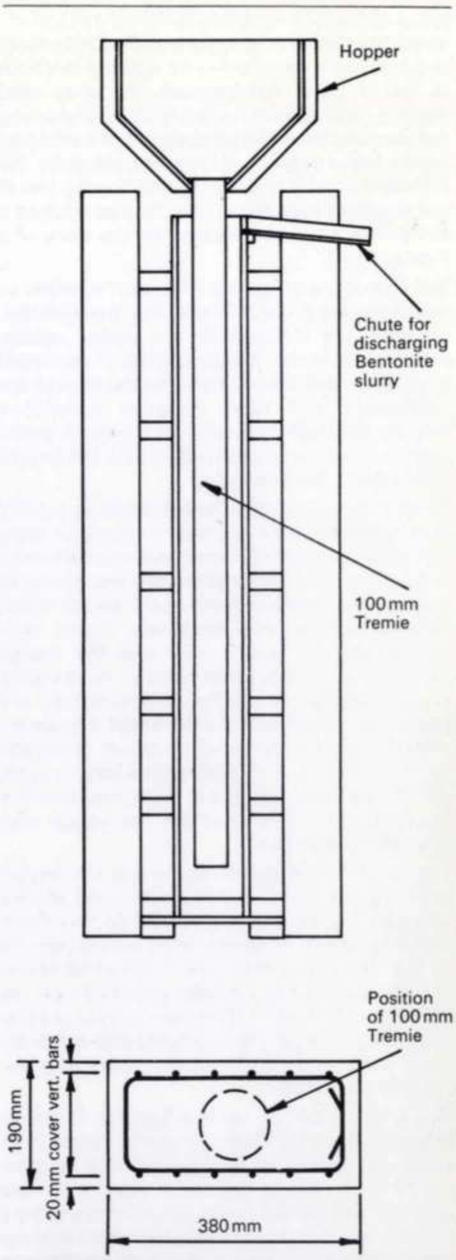
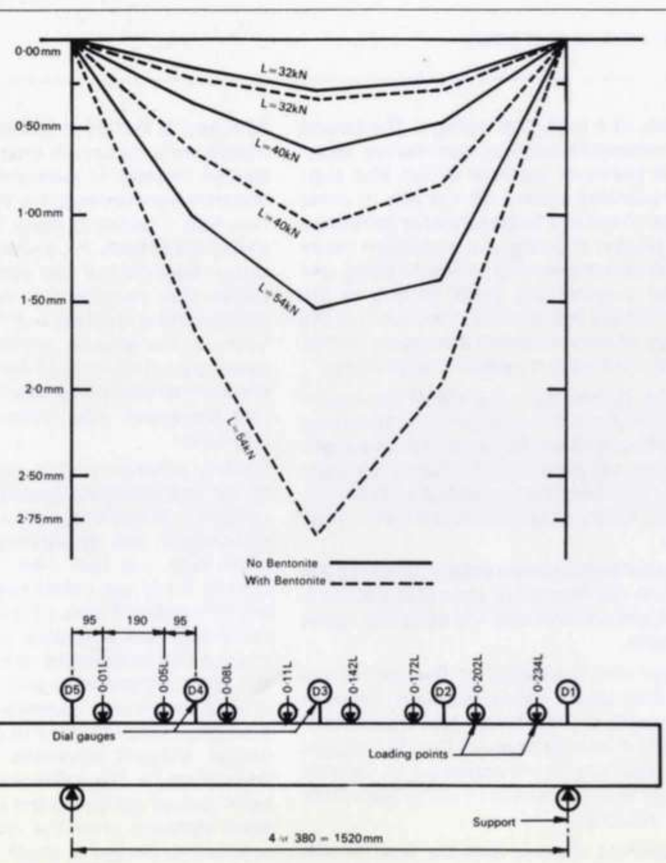


Fig. 5 *right*
Section through formwork and Tremie and, below, detail of test panel

Fig. 6 *below*
Above, deflections of test beams: below, test beam with loading points and dial gauges



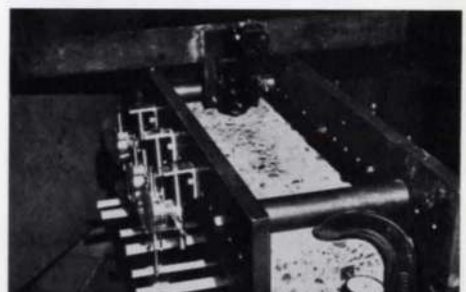
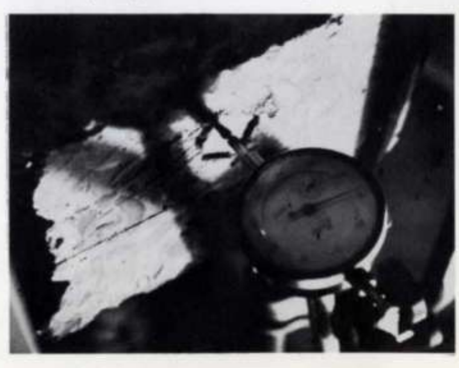


Fig. 7
Specimen beam, cut from diaphragm wall, in test rig

Fig. 8
End of test beam from diaphragm wall in test rig



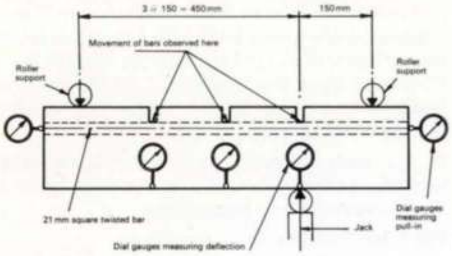


Fig. 9
Test arrangement for specimen beams cut from diaphragm wall and control beam

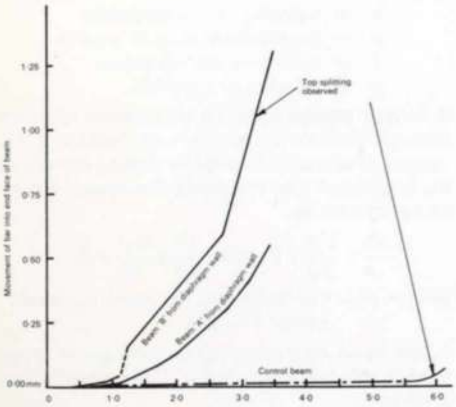


Fig. 10
Specimen beams : reinforcement slip at end nearest load

specimens were sawn from it. A control beam was cast in a mould with reinforcement and concrete strength similar to the specimens.

The beams were put in a test rig and instrumented so that deflection and slip of reinforcement could be measured as the test load was applied and increased. The results were convincing ; the bar in the 'control' beam split the concrete before slipping appreciably whereas the bars in the specimens cut from the diaphragm wall pulled in visibly at less than half the stress.

The case of the collapsed domes

At another time in the past, I was asked by one of the Senior Partners whether I thought that it was within my brief to go and look at some collapsed lattice domes in a far north location. I replied that whether or not it was part of my brief, I wanted to see them and made the necessary travel arrangements.

The domes had apparently collapsed at two-hourly intervals during a blizzard which had not been particularly violent as far as wind speed was concerned, but which had carried rather a lot of wet snow. The appearance on site was quite startling with the windward half of the domes standing intact and the leeward half looking more like a crashed airship than anything else.

Looking a bit closer at the debris, it appeared that the majority of the members were undamaged along their length and all the fractures had occurred at the connections. The connections were seen to be very ingenious from the point of view of production and erection, but they had a number of undesirable features with regard to stress transfer.

The geometry of the connection details was such that an accurate stress analysis by calculation appeared to be impractical and we therefore had samples of the joints taken from those portions of the domes which were still standing, assuming that the connections in these parts would not have been distorted by the collapse of the other half of the domes.

To establish the ultimate strength of the connections, we had the samples tested in tension and in compression with the load applied axially and with a slight eccentricity to ascertain the effect of any possible bending moments in the members.

The strengths of the connections were compared with the forces calculated by a space frame analysis of the lattice. The design brief had specified a snow load of 15 lb/ft² in accordance with *CP3 Chapter V*, and we found that the factor of safety against collapse under this snow load was less than that specified in the code of practice in force at the time of the design. But nevertheless it was enough to ensure that the domes would have withstood this snow load if it had been applied uniformly over the whole of the dome.

Domes and arches rely on their shape to resist load by axial forces in their members rather than by bending. For such structures, partial loading can produce worse conditions than overall loading, so the thought occurred to me that snow load over half of the dome might have been more dangerous that the same intensity of load applied to the whole of the dome.

As it happened, eye witnesses had testified that the snow had piled up on the leeward halves of the domes prior to the collapse whereas the windward sides had kept relatively free of snow. (After the collapse of the first dome, an attempt had been made to clear the accumulated snow on the second and in the process, the thickness of the snow had been measured and a cube of snow was cut out and weighed to get some idea of the density).

A further analysis was therefore carried out and this showed that 15 lb. snow load over only one half of the dome drastically reduced the factor of safety and after further refinement, it was shown that a snow load of a maximum intensity equal to that measured on site and of similar distribution would produce forces in the members which exceeded the ultimate strength of the connections.

Moral : look for simple explanations

I think if there is any moral to be drawn from these examples, it is : firstly that observance of the codes of practice does not ensure freedom from collapse, and secondly, that the basic cause of a failure is usually seen in retrospect to be fairly obvious and fairly straightforward, and the detailed analysis is really only used to prove the point, not to make it in the first place.

(All photographs in this article are by Poul Beckmann)

Fig. 11
Inside view of collapsed dome

Linear air distribution: principles and practice

Ken Aldridge

This paper was first published in the November 1972 issue of The Building Services Engineer, *the magazine of the Institution of Heating and Ventilating Engineers.*

Introduction

Most products are developed to satisfy a need and so it has been for air terminal devices, from the very basic mesh to conclude an air vent or duct to the more sophisticated square, rectangular or circular diffuser.

With standards of comfort increasing, the need came about for a terminal device to handle fairly large air quantities at low noise levels and provide the best possible air pattern, especially when supplying cold air with temperature differential up to 11°C. This was answered by the linear diffuser, which was initially produced in America. These diffusers have now been available in this country for many years with all shapes, sizes and designs to choose from.

With the advent and general acceptance of open plan design, the demand in terms of supply and technical performance for this type of terminal device has increased. In fact, for any good quality air conditioning design, linear diffusers are becoming the rule rather than the exception. It is the intention of this article to outline the basic principles of linear air distribution and describe the flexibility of this system, especially the most successful vane controller designed on air flow principles utilizing deflecting members, particularly the Coanda surface.

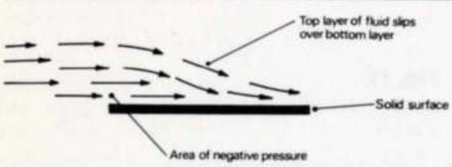


Fig. 1

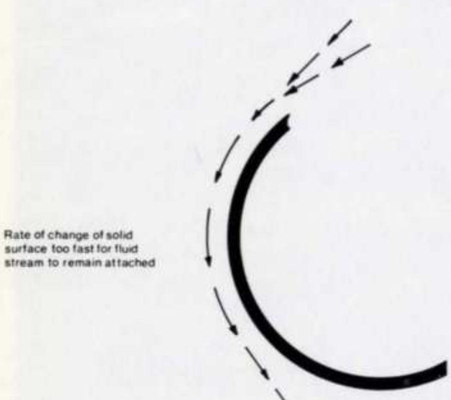


Fig. 2

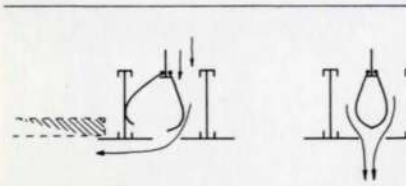


Fig. 3

Background

The name 'Coanda' is now becoming common so perhaps a little explanation may help.

Henri-Marie Coanda, a Rumanian, was born in Bucharest on 6 June 1885. His qualifications include a doctorate in engineering with degrees in aero-engineering, electrical engineering and refrigeration. He worked with the Bristol Aircraft Company in the early years of this century. His work on fluid flow across deflecting members contributed to the development of the aerofoil and it was from this that Coanda discovered the phenomenon of a fluid in motion naturally tending to cling to a solid surface and to change its direction in relation to that surface. This phenomenon can be visualized by remembering how a jet of water from a tap, although having a force and direction downwards, will immediately run down a person's arm if the hand only touches a small part of the water stream. It is this natural attraction of a fluid in motion to cling to an adjacent solid which is called the Coanda effect.

As the fluid attaches itself to the surface, the top layer of the fluid begins to slip over the front of the stream as the lower layer is reduced in speed by the friction between it and the surface. With the slipping of the top layer a negative pressure is produced which pulls down the top layer nearer to the solid surface and the whole process is repeated once again. This results in the fluid stream thinning out and for a certain period during the process the velocity of the fluid is increased quite substantially. The velocity increase is simply due to the normal mass flow laws being followed (see Fig. 1).

Coanda used the creation of the negative pressure to obtain the necessary lift of the aerofoil section whilst others have continued to determine the optimum shape for a maximum rate of change of direction of the solid, in relation to the original position of contact, without the fluid stream and the solid surface parting company (see Fig. 2).

For air distribution purposes the aim is to keep the mass flow/surface area ratio of the supply air as small as possible. This will ensure good control of throw and drop and provide vertical air patterns commensurate with comfort conditions. The linear air diffuser can provide all this, especially the type having the control vane designed on the optimum rate of change of direction. Similar results can be achieved by flat vanes, but pressure drop is increased and flexibility is reduced. A single slot diffuser with the specially designed deflecting vane can be seen in Fig. 3. A full 180° range can be chosen by simply adjusting the vane.

One of the important aspects of this 180° air turn is that horizontal air throw can be achieved. With the average type of diffuser, whether linear or otherwise, to obtain horizontal projection the ceiling is used, again in the Coanda fashion. The comparatively thick wedge of air emanating from the diffuser is lifted towards the ceiling and reasonably good comfort conditions can be maintained with an 11°C supply air differential. However, with the use of many types of coffered ceiling arrangements and the like, there is no ceiling on which the air can cling; this results in dumping. With this optimum guide vane shape, the air is turned 90° and remains in that direction with or without a ceiling surface.

By the time the air has turned to a horizontal position it has thinned out to nearly 1 mm in thickness and may be travelling as fast as 60 m/s without the noise level being above NR35. This very thin wedge gives the low mass flow/surface area ratio of supply air that is required and, because it is thin, early temperature equalization takes place and air movement is maintained by the force of the supply air rather than by the density difference (Fig. 4).

Performance

The performance characteristics of linear diffusers are inseparable from the characteristics of the header box or duct which feeds them. Consequently it may be in order at this point to remember that air is subject to the laws of fluid flow and that the design of all ducts and headers must be based upon Bernouilli's theorem which states that:

In any body of flowing fuid, in the absence of friction or other work effects, the total head is constant along any stream line.

Put into the form of an equation:

$$H_T = \frac{P_1}{\rho} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\rho} + \frac{V_2^2}{2g} + Z_2$$

where H_T = total head
P = pressure at a given point in the airstream
V = velocity of the airstream
g = acceleration due to gravity
Z = elevation above datum
ρ = density of the fluid.

It further asserts that the dissipation of work through friction reduces the total head and the receipt of work (as in a fan or pump) increases the total head. Consequently the equation may be re-written as:

$$\frac{P_1}{\rho} + \frac{V_1^2}{2g} + Z_1 = W + \frac{P_2}{\rho} + \frac{V_2^2}{2g} + Z_2$$

where W = the head loss (or gain) between points 1 and 2.

Since head multiplied by density gives pressure, multiplying both sides of the equation by ρ gives:

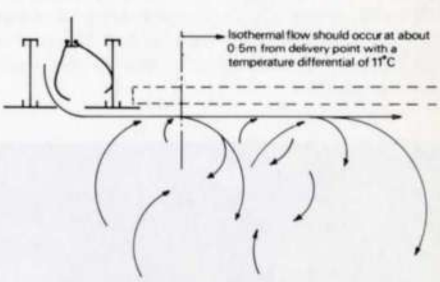
$$P_1 + \frac{\rho V_1^2}{2g} + \rho Z_1 = W + P_2 + \frac{\rho V_2^2}{2g} + \rho Z_2$$

where P is the static pressure SP, $\rho V^2/2g$ is the velocity pressure (VP) and ρZ the elevation term.

If we are dealing with a horizontal duct $\rho Z_1 = \rho Z_2$ and cancels, so we can rewrite the equation:

$$SP_1 + VP_1 = W + SP_2 + VP_2$$

If we consider a length of ducting of constant



Isothermal velocity lines showing how the force of the air supply jet entrains and discharges and thereby mixes room air

Fig. 4

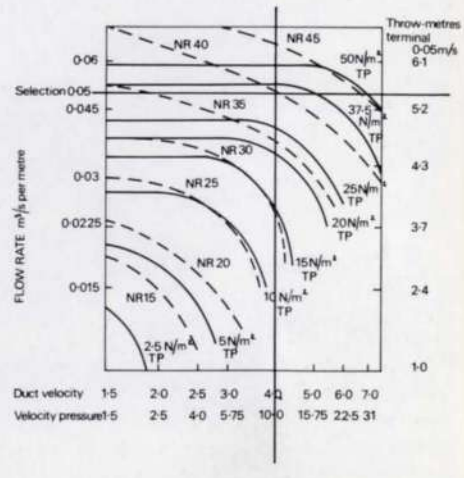


Fig. 5

cross-section, along which air flows at a constant velocity, then the energy dissipated by the air in friction at the duct walls shows up as a loss of static pressure between points 1 and 2. This has led to design principles being stated in terms of static pressure loss. In actual fact, the loss is one of total pressure, and design methods utilizing, for example, static regain still involve a total pressure loss; all that is achieved is a more suitable balance between the static and velocity pressures if the elevation term is neglected.

When designing header boxes and ducts for use with linear diffusers, it is important that a correct balance between static and velocity pressures is maintained and the most satisfactory way of achieving this is to use the ratio of the total pressure (the sum of the static and velocity pressures) and the velocity pressure as a criterion of correct balance.

The performance characteristics for a typical linear diffuser are shown in Fig. 5. These curves are for a single slot 'T' arrangement having a 20 mm air diffusion slot.

Considering a particular air volume per metre run of diffuser, it can be seen from the diagram that constant horizontal pressure lines exist over a certain velocity range within the header box or duct. This is the most stable position. As the constant pressure lines curve downwards, increasing variation in the air distribution rate along the slot can be expected. If the design point falls below the end of the constant pressure line, part of the slot will not be utilized.

It can also be seen that as the design point is moved along the horizontal part of the constant pressure line, varying velocities can be obtained, together with a variation in the regeneration noise level.

Most manufacturers' performance curves are based upon a certain set of standards. Care should therefore be taken on the selection of the equipment, and the variations in the standards that must occur for a specific design must be taken into account. Quite often correction factors are provided, but, for guidance, the main set of standards generally used is given below:

(i) Standard air (ie, barometric pressure, etc)
(ii) Throw rate of a particular length and size of slot
(iii) Throw rate for a particular air terminal velocity
(iv) Throw rate for a particular direction of throw (ie, horizontal or vertical)
(v) Room air to supply air temperature differential (positive or negative)
(vi) Type of header duct inlet (ie, centre-fed or end-fed)
(vii) Regeneration of noise levels based upon specific acoustic room absorption.

Most of these variations do follow mathematical laws, but it is as well to consult the manufacturer concerned if any doubts exist.

When a design point has been selected for a specified air volume rate per metre run and a specified noise level, consideration must be given to providing the correct air pressure and velocity coming onto the diffuser. The total pressure indicated from the curve is the most important aspect and must be used as a basis for any duct sizing. Details are given later for each type of header arrangement, but as a general rule the total pressure/velocity pressure ratio must always be more than three to maintain balanced conditions. Therefore, so long as the total pressure requirement is adhered to, variations in velocity, which might be imposed by space limitations, can be tolerated.

Aesthetics

Once the decision has been made that linear air distribution is required for a particular project, certain basic steps of design selection should be taken, not only to achieve the optimum in performance, but also in economic terms.

In designing an air conditioning system, one usually sets about the task by analysing and computing all types of heat gain and loss to a space, so that the space can be controlled to within predetermined limits relative to the financial resources available for the system and the function to which the space is to be put. From this the air volume can be obtained and with complete knowledge of the building and the proximity of the proposed plant room, the type of air transportation can be decided, ranging from high to low velocity.

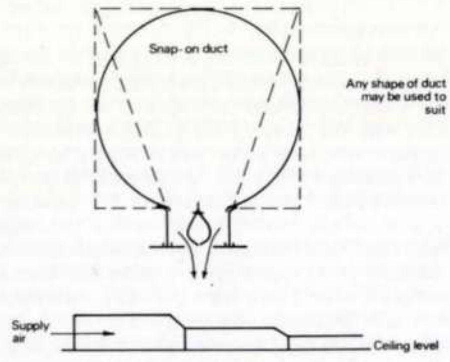


Fig. 6

Other requirements that must be known or assumed before selection can begin are:

(i) maximum supply air to room air temperature differential for cooling
(ii) Maximum background noise levels acceptable from the air conditioning system
(iii) The method of linear distribution.

Item (iii) is the decision that is often guided by the financial aspect and the architectural involvement in the scheme. The aesthetics of the space to be air-conditioned are not usually in the hands of the engineer. The resulting design is often a compromise between the technical advice and the architectural considerations, closely followed by the financial side.

Some of the considerations are:

(a) Any ceiling module that has to be adhered to
(b) The anticipated lighting layout and type of lighting fitting (ie, recessed, flush, etc.)
(c) Width of air distribution slot acceptable
(d) Does the architect require a completely clear ceiling for architectural freedom?
(e) Linear air slot part of lighting fitting – walls, partitions, furniture.

Most problems of this sort are peculiar only to the project under consideration and although there are a considerable number, most can be dismissed in the general design process which should then leave a solution which is practical to both the architect and the engineer. However, the initial choice of linear air diffuser layouts is generally made by the engineer and eventually given to the architect for approval. If the architect does not favour that particular solution then a compromise will have to be carried out. This method of approach may seem rather long-winded, or indeed unnecessary, but as air terminal devices play an important part in the appearance of a design, a little time in this direction may save quite a number of redesigns all the way back to the plant room.

Header types

Regardless of the way the air is taken to the space to be controlled, the final delivery to the linear air diffuser must be related to the conditions required in the room. However, the method of connection to the diffuser can be varied to suit the ceiling void depth and to a certain extent the supply air pattern. There are three basic methods:

snap-on ducts
header boxes
and pressurized voids.

(i) The snap-on duct type is shown in Fig. 6. The sizing of the duct in this application still requires a minimum *TP/VP* ratio of three and for ducting economy it is usual to let the ratio rise to approximately 10 and then reduce the duct size to give a *TP/VP* ratio of four or five and then allow once again the ratio to increase as the volume of the air in the duct reduces. The maximum overall length is often determined by the maximum size duct that can be handled. For an example on sizing take the requirement of 0.5 m^3/s to be distributed along a diffuser 10 m long with the entire length active. This gives 0.05 m^3/s per metre run which we can assume will give us our required noise level and throw.

The performance curve shown in Fig. 5 gives an initial total pressure requirement of 30 N/m^2; therefore our first duct size can be determined, since our maximum duct velocity must be computed from the maximum velocity pressure, equal to one-third of the total pressure, which is 10 N/m^2. This gives a velocity of 4 m/s and a duct diameter at entry of 0.385 m.

The next step is to determine where a reduction in duct size can take place. Although it is normal to reduce the size when the *TP/VP* ratio has increased to 10, it is as well to look at duct lengths, manageable segments of ducting and installation ease as well as duct economy rather than keep to a specific ratio.

Assuming that in this instance the best compromise is 10, then we can proceed and determine the second duct size. For simplicity of calculation and because the duct velocity is extremely low, it is easier if the friction loss along the ductwork is ignored and that our total pressure along the complete length of the duct and diffuser is taken as constant. Therefore the velocity in the duct when the *TP/VP* is 10 is $\simeq 1.28\sqrt{TP/10}$ which gives 2.2 m/s or 0.25 m^3/s.

Thus, by bringing back the *TP/VP* ratio to five, we require a reduced duct diameter

$$\simeq \sqrt{\frac{4\times m^3/s}{1.28\,\pi\sqrt{TP/5}}} \simeq \sqrt{\frac{4\times 0.25}{1.28\,\pi\sqrt{30/5}}} \simeq 0.32\text{ m}$$

This can be carried out until the complete length is sized. If the duct reductions do not fall easily, then it is a simple matter to start along the duct run once more and manipulate the *TP/VP* ratio as necessary. Balanced conditions will always exist providing the *TP/VP* ratio does not fall below three.

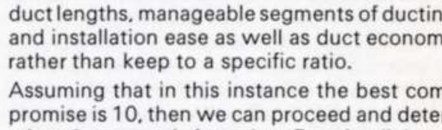


Fig. 7

The air pattern or direction of throw relative to the duct axis will vary slightly with the *TP/VP* ratio (Fig. 7). This can, of course, be used to some advantage but generally does not affect the overall room air pattern. With the ratio at three, the direction of throw is approximately 60° at the duct axis, gradually straightening to 90° at a ratio of approximately 15.

(ii) The header box can either be a standard manufactured box or one designed by the engineer as in Fig. 8. For small air volumes

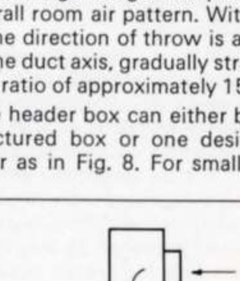


Fig. 8

standard products should be available, but once the air volume to be handled becomes excessive or unusual space limitations exist, then headers have to be specially designed.

The sizing principles used on the snap-on duct still apply; the only difference is the shape of the duct or header. Quite often the void space allocated for services is extremely small and this is sometimes aggravated by low noise requirements.

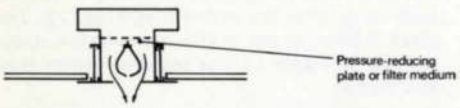


Fig. 9

Should a header duct be reduced in size, this will obviously produce a higher velocity pressure than is actually needed. Therefore to maintain balanced conditions the total pressure must also increase (see Fig. 9). The result will be to increase the noise level which originally related to the selected volume flow rate per metre run of diffuser. To overcome this difficulty, the header duct and the top of the diffuser can be separated by a perforated sheet or filter medium to absorb the additional pressure. The sheet should be used if only a small pressure drop is required, whilst for high pressure drops of up to 370 N/m² a filter medium may be chosen. With the filter medium, it is then worthwhile to include a secondary filter in the main duct system to ensure that the air passing through the linear diffuser is as clean as possible. Furthermore, in the event of space control being especially important, an alarm signal on the secondary filter when the pressure drop rises to a certain limit is advisable. Without this precaution there will be a gradual decrease in the air supply as the secondary or linear filter medium becomes clogged.

The horizontal air pattern for the header box behaves in a similar way to snap-on ducts if the run is continuous. With the small standard plug-in boxes, however, the air pattern corresponds to the shape of the box itself. With sloping ends the air pushes out at an angle whilst for a square box the air distributes at right angles (see Fig. 10).

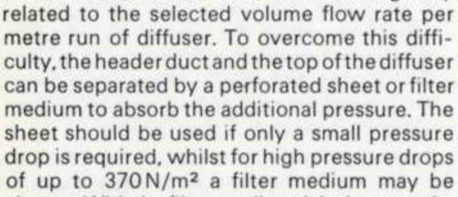


Fig. 10

(iii) The pressurized void system is extremely popular when minimum space exists within a ceiling void and the lighting load or sensible gain to the ceiling void is small in relation to the total sensible gain.

The ceiling void itself acts as a header duct, but to ensure balanced conditions over a number of linear diffusers within the ceiling, the velocity pressure at the first diffuser, with respect to the air flow, must be no more than one-tenth of the pressure drop across the diffuser. This ratio has been determined from empirical work and applies whether the ceiling void is used for supply or extract purposes. So long as the ratio is one-tenth or less, the system becomes self-balancing and the performance figures of the particular linear diffuser apply, the only difference being that the header duct has been removed and the *TP/VP* ratio increased.

The use of the ceiling void for air distribution is fairly new in comparison to suspended ceiling systems and therefore normal types of mineral fibre, wood fibre and metal ceiling tiles have not been designed for this application, except for ventilated types.

The air leakage through some types of tile is quite considerable and care should be taken in tile selection otherwise results in air distribution may be disappointing. Some manufacturers now provide the facility to seal the tile by applying a metal foil to the back. It is also recommended that a concealed fixing ceiling system is used to limit still further tile edge leakage. The air pattern from this application is similar in every respect to that described for the snap-on duct. There is no particular limitation in the length of ceiling from the supply duct; good results have been achieved up to 35 m. One point to bear in mind when designing the system is that very often the ceiling void has obstruction such as beams, lighting fittings and possibly other ducts, so the limiting velocity (for the correct *TP/VP* ratio) must be at the greatest obstruction passing the greatest amount of air.

Horizontal air patterns

Linear air distribution gives its greatest flexibility in the type of air pattern and positions of supply points that can be used.

Arrangements usually comprise a series of linear diffusers stretching across the room, corresponding to a building module, a ceiling tile module or lighting layout. This is done to achieve a neat appearance which often means that there is more diffuser slot available than is actually necessary for air distribution. Thus a choice is available for the position of the active air sections which can be chosen at the commissioning stage when greater knowledge of the positions of high sensible heat gain, due to machinery or occupants, are known.

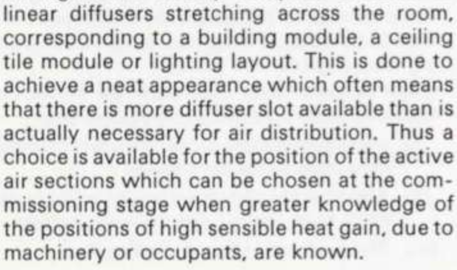


Fig. 11

Fig. 11 shows a simple arrangement of linear diffusers. The total lengths of diffuser often have the vane controller split down into 300 mm sections which enables the supply air direction to be staggered. The air patterns shown are for horizontal projection.

Occasionally, a design may require that the air shall be supplied from one side of a room, possibly because the architect or interior designer wishes to create a special ceiling which is in no way linear. This can be achieved quite easily for air throws up to about 20 m with a limiting NR of 35. With this type of supply the comfort conditions are in no way reduced, in any part of the room, from the high standards that are obtained using a symmetrical distribution.

The volume flow rate per metre run requirement is greatly increased, but the problem of noise is solved, by using a multi-slot arrangement as in Fig. 12. This method can often create an interesting edge detail if it is included with a ceiling upstand incorporating a curtain track or lighting wallwash. Because the thickness of the air emanating from the slot is thin, with a multi-slot arrangement it is possible to layer each air stream so that collision and subsequent down draughts do not occur.

One technique developed from the multi-slot arrangement is to motorize the air diffusion vane nearest the wall or window if that wall or window is contributing a high heat gain or loss to the space. The air supply on that one slot can be directed towards the wall and counteract sensible gain or loss. The motor can regulate the quantity of air in relation to the internal surface temperature so that down draughts are prevented and the local air temperature maintained. This method has been used on a number of projects quite successfully.

High air-change rates

The linear diffuser system can handle most air change requirements. Projects have been successfully designed for computer suites with air change rates over 120 per hour.

The most common method of distributing the air in computer suites is the perforated ceiling system using a positive pressure void. Air is then extracted at low level, or in floor grilles if a suspended floor has been installed. Whilst the perforated ceiling system functions extremely well in this and other applications, it will only work well if the extract point is at low level. High level extract points can cause short circuiting due to the natural convection currents, from the areas of high sensible gain, being higher in pressure than the air supplied from the ventilated ceiling.

If it is not possible to have a low-level extract point, the pressurized void system can still be used using the linear diffuser. The void acts as the header duct and the high air volumes can be handled by lapping the air streams as shown in Fig. 12. The high air velocities used ensure that good balanced conditions exist and will not be affected by the position of the extract points.

Variable volume

When a variable volume system has been chosen, troubles often occur at points of minimum volume, causing certain areas to become stagnant, whilst others suffer from drop. To overcome this using conventional square or circular diffusers, the number of diffusers has to be increased to give good coverage and to limit the range they have to handle. This results in a very messy ceiling arrangement with extra expense involved, both in terms of equipment and labour.

Since the linear diffuser has a very low volume/surface area ratio, its coverage of the space to be controlled is extremely wide. Also, with the diffusion vane having a Coanda surface, no drop of the air will occur at any point between 0 and maximum air volume. Moreover, if individual air patterns are designed to overlap at maximum volume and meet at minimum, volume, complete coverage of the space is achieved at all times (see Fig. 13).

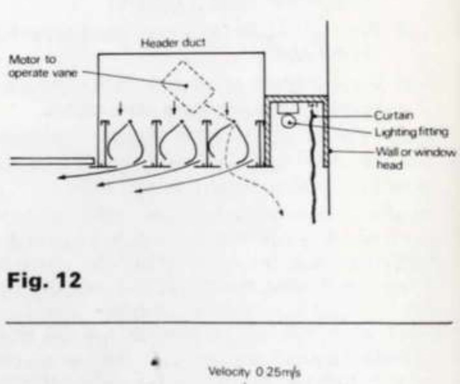


Fig. 12

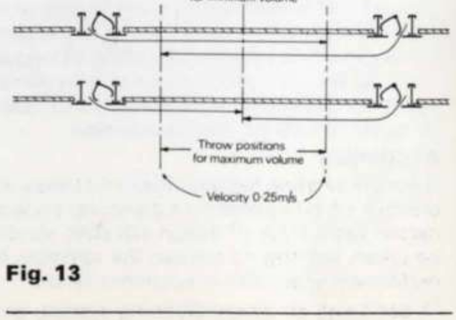
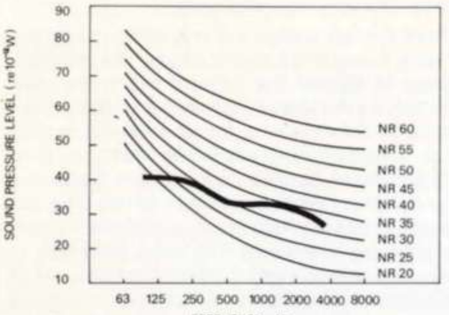


Fig. 13



The heavy curve shows the level of regeneration (NR 35) that occurs with 12mm slot in a linear diffuser under an air pressure of 42 N/m² (total) giving 0·5m³/s. This curve can be moved up or down by changing the volume flow rate or dampening to give a higher back pressure.

Fig. 14

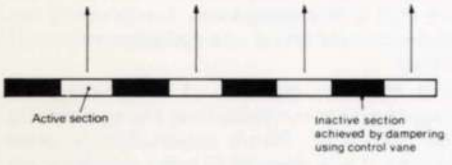


Fig. 15

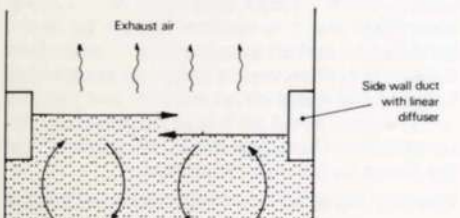


Fig. 16

Tuning

Now that the era of glass buildings is slowly disappearing, some buildings are being designed to give an external fabric sound attenuation around 45 dB to nullify the noise-polluted atmosphere that occurs near airports, high density traffic areas, etc. With open plan offices and the high degree of soft furnishings, the acoustic atmosphere, between the passing of aircraft or a lull in the traffic, becomes dead and feels uncanny to most occupants.

Sophisticated electronic noise can be used to maintain a specific background noise level, but if a building is designed to keep out the noise it has to be relatively airtight and therefore mechanical ventilation or air conditioning is installed. Therefore, the air distribution equipment can be selected to provide the necessary background noise level. This can be obtained and regulated from the regeneration of noise that occurs as the air passes through the diffuser. It is recommended that, if this method is going to be used, the volume flow rate per metre run and air pressure rating be chosen so that the noise level for the duty is at least 3 NR lower than is actually required. Also, the performance point should be favourably balanced. Alternatively, the length of linear diffuser required to handle the required air volume should be about 20% more than is required. This ensures that for the same design header pressure the volume flow rate per metre run is reduced if all the diffuser remains active, thereby reducing the noise rating. The margin in noise level allows for any discrepancies that may occur in the acoustic design in relation to the acoustic environment that is eventually built. Thus a positive or negative tuning of the background noise level may be carried out at the commissioning stage to suit the actual conditions. With the Coanda diffusion vane the sound spectrum of the regeneration follows closely the NR curves. This gives much better control of the acoustic environment, especially if masking is required (see Fig. 14).

For an example, consider a room requiring 0.5 m³/s, having a background noise level of NR 35. The performance curve shows that an active distribution of 0.05 m³/s per metre run is required and therefore a length of diffuser 10 m long. For this exercise, if we size on 0.05 m³/s per metre, but install 20 m of diffuser, this allows for 50% of the installed length to be shut off (Fig. 15). If the performance point selected is perfectly stable, it is possible to shut off or open other sections of the diffuser on site within a range and thereby change the regenerated noise level, up or down, without affecting the total volume of air that is required to be supplied to the room. Within the range the air throw will vary only slightly.

High ceilings

With air conditioning systems being installed in old or lofty buildings, it is sometimes found necessary to move the total volume of the room air to maintain conditions. With a ceiling height from 6 m upwards, ceiling air supply becomes difficult and supply from the sides is limited by length of throw and drop, with the result of very uncomfortable air movement and temperature variation within the space.

One way to overcome the problem using a Coanda linear device (if a suspended ceiling is not being used) is to throw the air from the sides and cover the completed area of the space. This then creates a dead zone above the air pattern and enables only the air below to be controlled. If the extract duct is above the air controlled zone the dead zone can be classified as part of the extract system, even though there may be some sensible gain from the lighting fittings at high level (see Fig. 16). This system will also work well with a low level extract.

Other uses

The use of linear diffusers can have other advantages. With a double inverted 'T' ceiling arrangement many other applications are available to the architect or building services engineer (see Fig. 17). The 'T' can be used for:

(i) Supporting of electrical power and the support of lighting fittings
(ii) Location for partition heads
(iii) Supporting sprinkler heads or smoke detectors
(iv) Different ceiling systems with varying linear extrusions (ie, concealed grid or laying grid system)
(v) A ceiling void extract system.

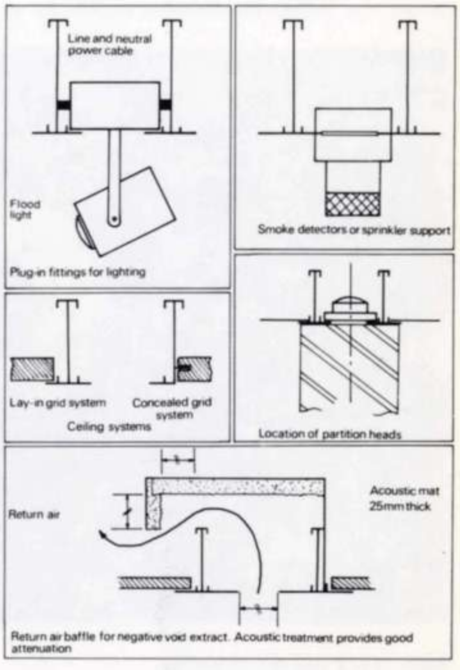


Fig. 17

Where cross-talk is a problem and a common void extract system is used, the static insertion loss can be improved by fixing a return air baffle lined with a fibre glass mat. This design will give a room to room attenuation of around 35 dB, providing the ceiling system and partitions are of the correct standard of construction and installation.

Acknowledgement

The author is grateful to Peter Platten for constructive suggestions made during the preparation of this paper.

J. G. Strydom Tower, Hillbrow, Johannesburg

Ron Finkelstein

Rising 269 m above the streets of Hillbrow, with the top of its mast almost 2134 m above sea level, the slender concrete pencil of the J. G. Strydom Post Office Tower in Johannesburg is the tallest structure in Africa.

With the development of high-rise buildings on the city landscape, the Post Office telecommunication links to Benoni, Carletonville and other stations were being interrupted. It was decided, therefore, that a tower of sufficient height to position the transmitting antennae above the tallest foreseeable buildings in Johannesburg should be constructed. The present siting of the tower was necessitated by the extensive existing cable system in the area of the present telephone exchange in Hillbrow.

The site chosen, while being technically correct, imposed unique problems for the designers of the tower. Being in a built-up area extremely prominent in the city, a civic responsibility existed to ensure that the structure was both acceptable visually and free from contemporary gimmicks which would become dated in the near future. The tower is thus built to remain prominent and acceptable into the 21st Century. The area of Hillbrow has a particular character of its own, consisting largely of rectangular, tall buildings, and the designers had to integrate the new tower into this environment. Of particular importance is the fact that the tower is masked by the various buildings at different heights when viewed from different parts of the city. Of prime importance, therefore, was the need to ensure that the proportions of the structure were acceptable under the varying conditions.

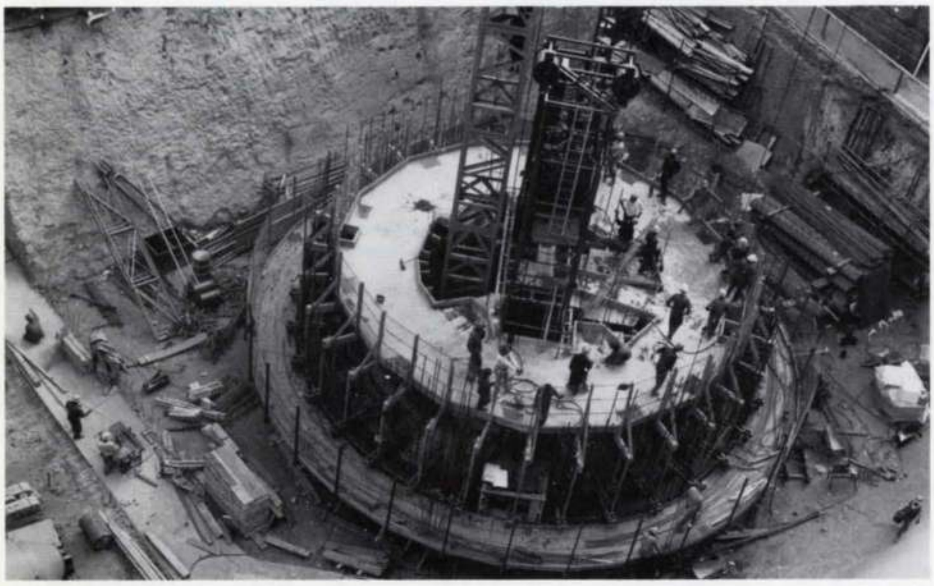
Three further design criteria were important. Firstly, being in a congested site, the available space to spread the base of the tower was limited. As the tower is governed both by stability and deformations, it was essential to provide maximum stiffness of the shaft, not only at the bottom, but over its full height. Secondly, the construction of a structure of this type can create extremely hazardous conditions for residents living in the area. The need, therefore, to choose a solution which limited the amount of movement of free formwork, etc, was extremely important. Thirdly, the tower had to be completed in an extremely short time, in order to prevent disruption of the city's telecommunication services. Simplicity in form was therefore essential.

Based on these various criteria the structural solution which now dominates the landscape was chosen.

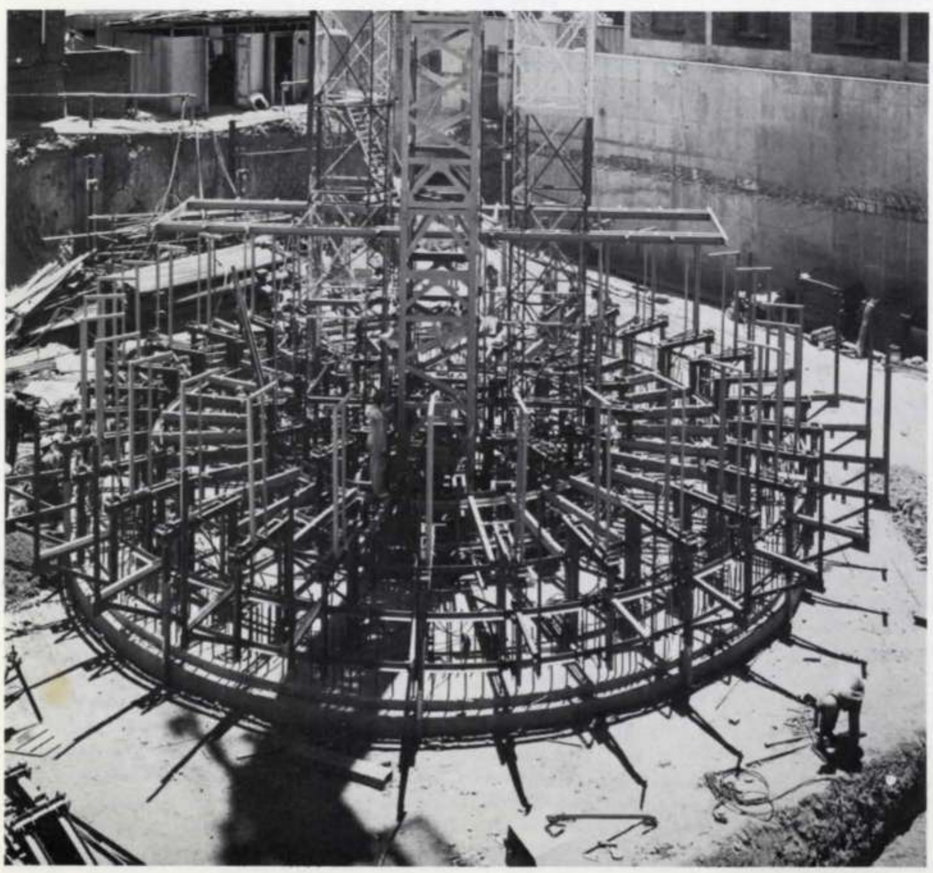
The first stage of the construction of the complex was to complete the tower to house the Post Office equipment. Concurrently the new automatic telephone exchange was completed.

With the main objective of constructing the upper turret in minimum time and to eliminate the hazards of difficult construction at great heights, it was decided to make extensive use of precasting. The two turrets, while being independent structurally in the completed form, had to be considered as a combined problem during the design and construction phase. The key section of the tower is the upper Post Office turret which houses extensive microwave equipment and this section had to be completed at the earliest possible time. The solution chosen, therefore, was to construct the skeletal frame of the lower turret in beam and column units, which acted as a bracing structure for constructing the upper turret. At a later stage the lower turret was completed.

Basically the structure comprises a reinforced



Figs. 1—3 above and below
Various stages in the construction of the tower
(Photos: Lynda Shave)



concrete tower built to a height of 232 m and topped by a 38 m high steel mast. The foundations of the tower consist of eight 3.2 m diameter shafts sunk to a depth of 42 m. The external structure is a free-standing cylindrical shell, 13.7m in diameter with parallel external sides top to bottom.

A transition in the structure occurs at 179 m where the 11-storey turret structure begins. The thickness of the concrete shaft at the base of the tower is 840 mm and then tapers to 380 mm at the top of the concrete shell. The service and lift shafts are constructed inside the structural shaft. The six lower public floors in the turret section have an external diameter of 20 m. Microwave equipment is housed on an additional two floors which cantilever out above the public levels.

To allow differential movement between the outer shell and the inner core and yet restrain plan rotations, a special sliding joint had to be incorporated in the transition stage. It was estimated that at the expansion joint there may be movements of up to 50 mm due to creep and shrinkage and 74 mm due to temperature fluctuations. The joint allows independent movement of up to 150 mm between the inner and outer shells, thus relieving the structure of longitudinal stresses.

In order to reduce the period of construction, the lower turret was constructed in large precast units consisting of columns and curved external beam panels. These units were assembled in advance of the floor construction and braced to the tower core. They were then used to support the heavy cantilever construction of the larger diameter upper Post Office turret floors, which were also made from precast wall units.

Particular attention was paid to the finish on these units which is a white exposed aggregate cast in white cement.

Computer programs were used during the design stage for solving the more difficult problems associated with wind vibrations and gust effects. One of the major design problems was that the slender nature of this structure made it susceptible to vibration under certain wind conditions.

It was calculated that the whole structure would have a low frequency of oscillation and it was designed rigidly enough to dampen any tendencies for the oscillations to become excessive. Vortexing was found to occur randomly at various heights on the structure depending on the wind velocity and one of the computer programs was used for comparing the energy impulse from the vortex shedding with the natural absorption of energy inherent in the structure.

Computer programs were also devised for checking the wind and eccentric load bending effects on the tower as a whole and also for determining quantities of reinforcement in the outer cylindrical shaft. The usefulness of having the design on computers was demonstrated when the overall height of the tower was increased at a very late stage. This necessitated a complete redesign which was carried out within a few days.

The inter-connection of the 38 m steel mast through the lift motor room provided unusual construction details, which were solved by means of an elaborate frame at the lift motor room level. Consideration was given to the erection of the mast by using a helicopter, but it was eventually decided that the time schedule could be met by means of conventional construction.

The construction was carried out under very exacting access problems and pressure from local residents to restrict noise and dust nuisance. The contractors worked closely with the designers and showed much ingenuity in methods of construction and in the maintenance of accuracy of the construction. This problem of vertical alignment during building was solved by using a helium neon laser rigidly fixed at the base of the tower and focused on an upper target. The construction of the cantilever floors and the turret was simplified by the extensive use of precast concrete and a preformed system of radial steel beams. One of the most difficult problems was to ensure an adequate fire resistance in the turrets and considerable work was done to ensure that a collapse would not occur if an accidental fire broke out.

Although the public will not be aware of it, the apex of the mast on top of the tower will sway 860 mm under full wind loading, the microwave room 480 mm and the revolving restaurant 410 mm. This limited sway is essential so that the aerial alignment with other microwave stations remain accurate to the required one-third of a degree.

Construction of the tower commenced in January 1968 and was completed in March 1971. Costing £1,100,000 it has a mass of approximately 19,050 metric tons. In its construction 9,175 m^3 of concrete and 1,315 metric tons of steel reinforcement were used.

The Post Office realized that this landmark could be a great attraction for visitors and has accordingly catered for them. In an imposing foyer a display of photographs and diagrams provide sufficient information about the tower to satisfy the most inquisitive visitor.

Two high speed lifts carry visitors smoothly through the heart of the central ten service floors of the tower at 6 m per second, to six public floors, providing a vast window on the whole of Johannesburg. The panoramic view encompasses the famous Witwatersrand goldfields in the south to the sweep of the Magaliesberg mountains in the north.

The restaurant at the upper level revolves silently once every hour, providing 200 diners with an ever-changing view of the city. Below the restaurant is the highest cocktail lounge in Johannesburg. The public floors are carpeted throughout and air-conditioned to provide maximum comfort at all times. The visibility from the public floors, on a clear day, is over 80 km.

Credits Client: SA Public Works Department.
Architect: SA Public Works Department.
Main contractor: Christiani & Nielsen.

Fig 4
The completed tower
(Photo: Lynda Shave)

A review of awards gai

1

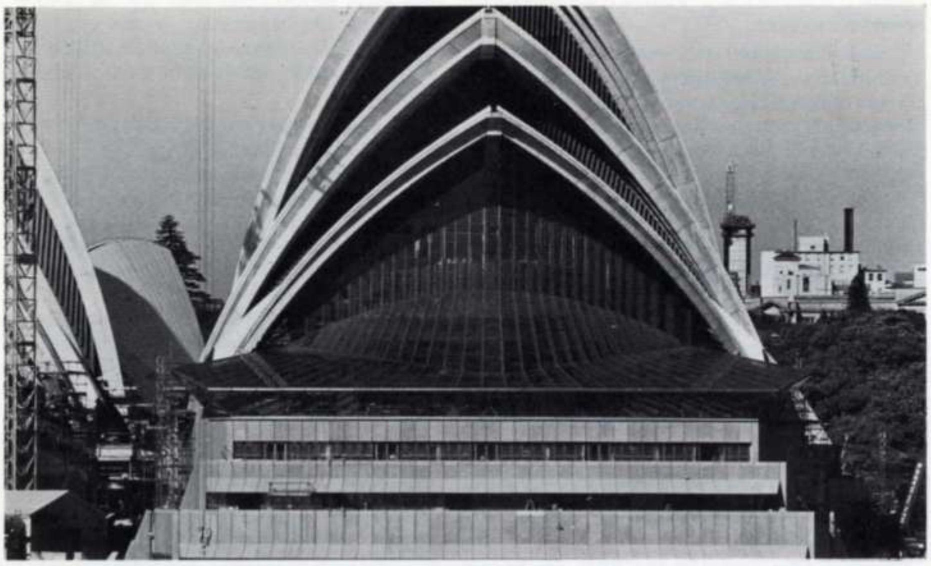


Photo: Harry Sowden

2

Photo: John Donat

ASSOCIATION OF CONSULTING ENGINEERS OF AUSTRALIA 1972 ANNUAL AWARD FOR EXCELLENCE

Job no. 1112—Sydney Opera House (glass walls) (Fig. 1)

Architects: Jørn Utzon (Stages 1 & 2)
Hall, Todd & Littlemore (Stage 3)

Glasswork sub-contractor: Boussois-Souchon Neuvesel, Ste.

CONCRETE SOCIETY AWARDS 1972

Highly commended

Job no. 2798—Crucible Theatre, Sheffield (Fig. 2)

Architect: Renton Howard Wood Associates
Main contractor: Gleesons (Sheffield) Ltd.

Special mention

Job no. 2102—Blackpool Police HQ and Law Courts (Fig. 3)

Architect: Tom Mellor and Partners
Main contractor: G. & J. Seddon Ltd.

Job no. 2859—Dalston County Secondary Girls School, Hackney (Fig. 4)

Architect: Jellicoe and Coleridge
Main contractor: Wates Construction Ltd.

Job no. 3429—Emley Moor Television Tower, Yorkshire (Fig. 5)

Designer: Ove Arup & Partners
Main contractor: Tileman & Co. Ltd.

DEPARTMENT OF THE ENVIRONMENT GOOD DESIGN IN HOUSING AWARD 1972

Highly commended

Job no. 3017—Garrad's Rd. Old People's Home, London SW16 (Fig. 6)

Architect: Lambeth Borough Architects Department

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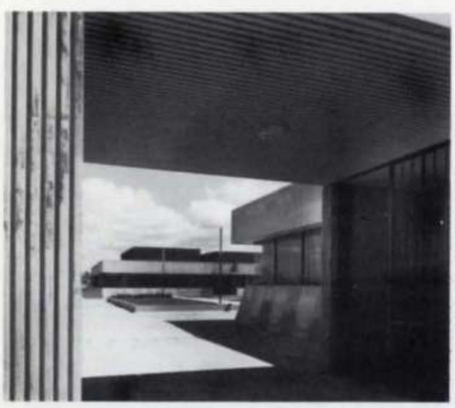

Photo: Norman Butler

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Photo: Sydney W. Newbery

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ed during 1972

FINANCIAL TIMES INDUSTRIAL ARCHITECTURE AWARD 1972

Winner

Job nos. AA152, 170, etc.—IBM Development, Havant. Phase 1, Respond Centre; Phases 2, 3 & 4, Systems Assembly Plant; Phase 5, Materials Distribution Centre (Fig. 7)

Designed by Arup Associates Architects + Engineers + Quantity Surveyors.

Commended

Job no. AA126—New offices and works for the *Oxford Mail and Times* Ltd. (Fig. 8)

Designed by Arup Associates Architects + Engineers + Quantity Surveyors

Job no. 3684—Studio for Bernat Klein at High Sunderland, near Selkirk (Fig. 9)

Architect: Peter Womersley.

THE RIBA ARCHITECTURAL AWARDS 1972

Job no. 1976—Birmingham Repertory Theatre, King Alfred's Place, Birmingham (Fig. 10)

Architects: S. T. Walker and Partners, in association with J. A. Maudsley, Birmingham City Architect.

Job no. 2062—Ulster Museum, Belfast. Extensions (Fig. 11).

Architects: Francis Pym, until March 1968; after this date the project was supervised by the Chief Architect's branch, Works Division, of the Ministry of Finance of the former Government of Northern Ireland.

Job no. 2798—Crucible Theatre, Sheffield (Fig. 2)

Job no. AA/175—'Horizon' Project. Factory for John Player & Sons Ltd., Nottingham (Fig. 12)

Designed by: Arup Associates Architects + Engineers + Quantity Surveyors.

SOUTH AFRICAN ASSOCIATION OF CONSULTING ENGINEERS AWARD FOR DISTINCTION IN DESIGN 1972

Winner

Job no. SA 979—J. G. Strydom Tower, Hillbrow, Johannesburg (Fig. 13)

Designed by: Ove Arup & Partners (South Africa).

Highly commended

Job no. SA 2043—Rosslyn Brewery, Pretoria (Fig. 14)

Architects: Powell, Whyte & Partners.
Prime agents: Ove Arup & Partners (South Africa).

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Photo: John Donat

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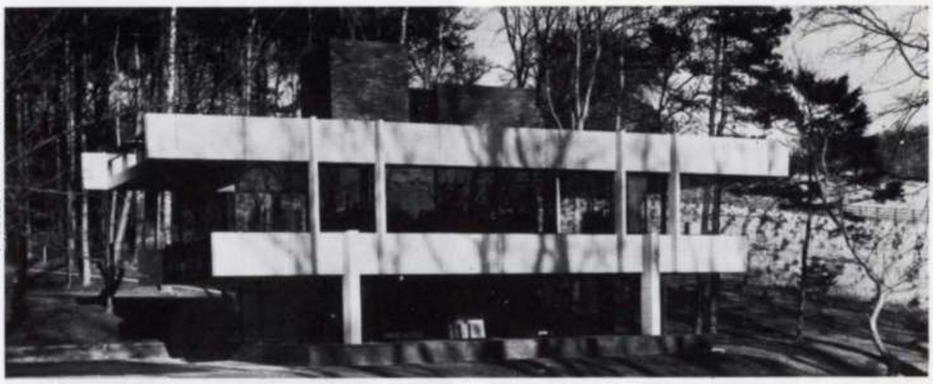

Photo: The Scotsman

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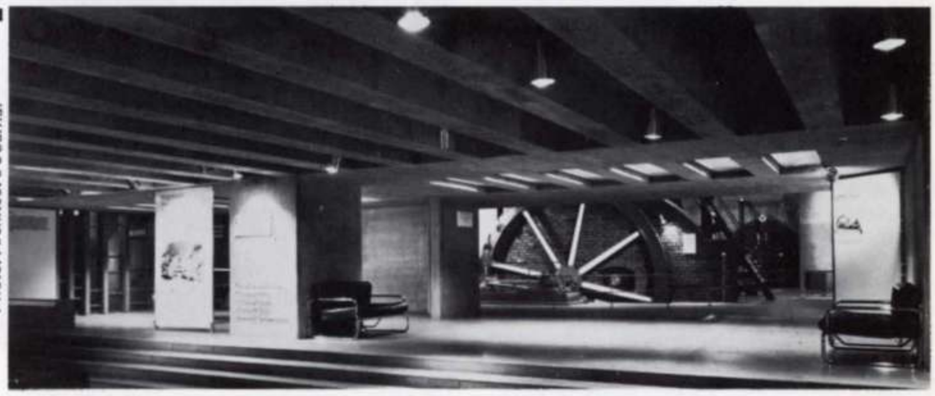

Photo: Architect's Journal

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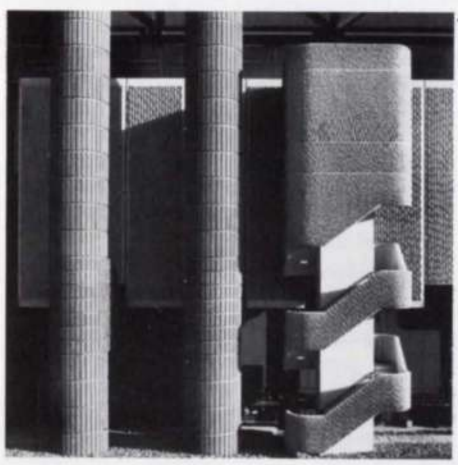

Photo: John Donat

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Photo: Lynda Shave

Jan Smuts Airport: New terminal buildings, roads and associated landside works

Geoff Bottom

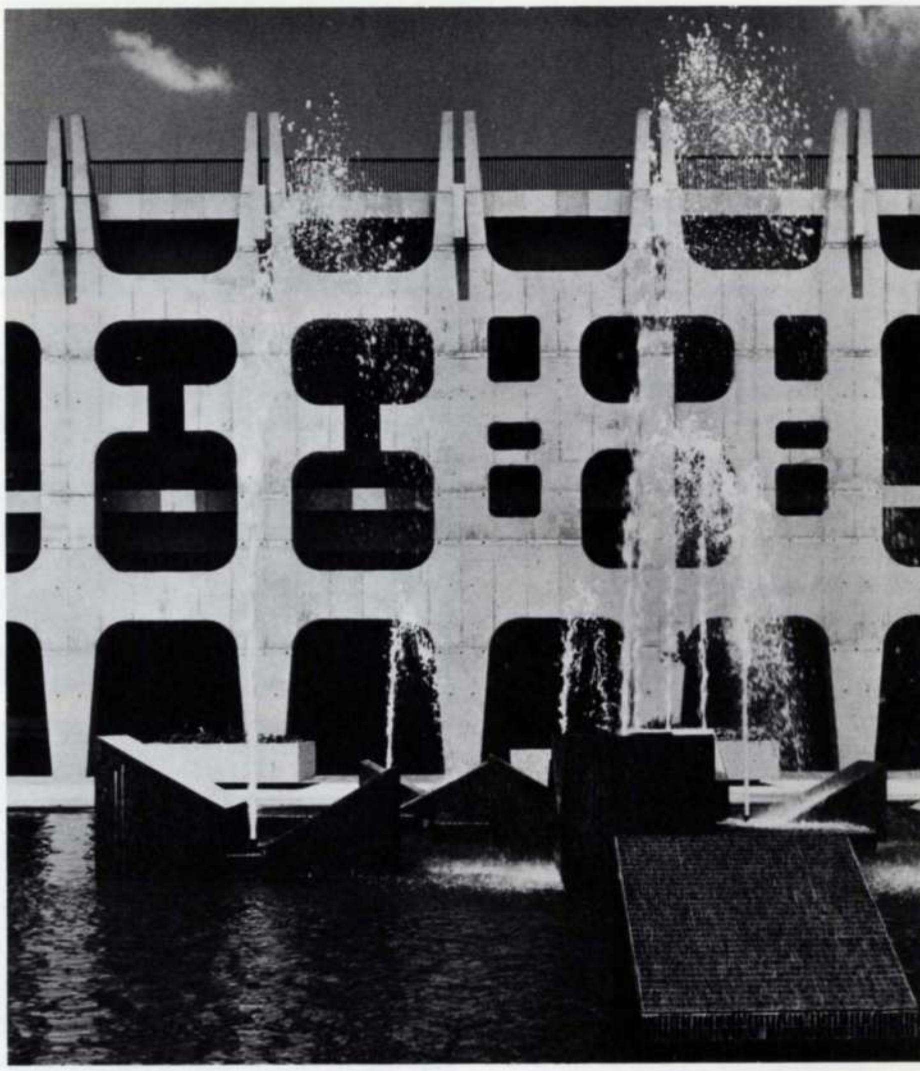

Fig. 1
Detail of ornamental fountain on forecourt piazza with façade wall supporting elevated motorway in the background

Introduction

At the beginning of February 1967, Ove Arup & Partners were appointed structural consultants for the complex of new terminal buildings at Jan Smuts Airport, Johannesburg. Such a complex had become necessary to meet the increasing demand for passenger and freight handling facilities at the airport and the brief demanded a tight and exacting schedule. It was essential that the construction programme was so phased that the new passenger facilities would become operational by the end of 1971, to coincide with the inauguaration of South African Airways' Jumbo Jet service to Europe.

The construction sequence for the terminal buildings was divided into five separate contracts to meet the building deadline and, throughout the various contracts, the entire design team was fully extended so as to remain one step ahead of the contractors. However, a close spirit of co-operation developed between the former and the latter as the building work progressed. The designers were thus able to adjust final details to meet on-site conditions and, as a result of this team spirit, the final building deadline was met.

The initial building programme comprised a terminal complex for international arrivals and departures at split levels with a separate domestic arrivals concourse. The concept for channelling arriving passengers to ground floor level and departing passengers to first floor required the incorporation of an elevated motorway as an intregral part of the design. These new passenger handling buildings cover a ground floor area of approximately 1.8 hectares, incorporating a basement over the whole area.

Located as an independent tower, an administration office block, seven storeys high with three basement levels, completed the brief for the initial building programme.

At an early stage in the project Arups were called upon to undertake a detailed traffic survey to predict parking requirements at the airport. Following a report to the Public Works Department, the firm was commissioned, as principal agents, to provide underground parking accommodation for 1,300 vehicles initially, to phase in with the opening of the new terminal facilities. The brief allows for the provision of parking facilities for 4,800 vehicles eventually, with the extensions being programmed for construction as and when required.

The parking garage comprises two levels of underground parking covering some 2.4 hectares of plan area and is linked to the terminal concourses.

The roof over the garage forms a paved piazza providing a forecourt to a strikingly patterned façade wall, which supports the elevated motorway to the departure concourse.

Associated with the parking garage, the firm was appointed civil engineering consultants for the design of the landside road network serving the terminal buildings and parking areas and linking up with the provincial roads interchange on the western boundary of the airport. The brief included all traffic surveys, control of traffic flow and parking for the final airport development.

The final aspect of the development for which the firm was appointed consultants was the provision of a 6.8 million litre reservoir and ring water main system for the building complex.

Terminal buildings

The site was officially handed over to the contractor on 1 June 1967 and some 87,000 m³ of material had to be excavated from the basement area before piling could commence. Piles ranged in diameter from 460 mm to 1.07 m and something over 1,200 piles were cast to form a foundation to the terminal buildings. All piles were taken down to refusal on a decomposed andesite lava.

The superstructure of the international terminal building and of the domestic arrivals concourse, together with the two adjoining buildings forming the main terminal complex, covers an area of approximately 240 m × 90 m. The international terminal building consists of a basement, a ground floor arrivals concourse, a departure concourse at the first floor and a roof which is used as a public viewing and waving platform. The head-room of the arrivals and departures concourses is in both cases 8.2 m. Two mezzanine floors are located between the main floors but cover relatively small portions of the building and, taking into account air-conditioning services, the headroom in these areas is reduced to approximately 2.9 m.

The main superstructure floors have spans varying from 8.2 m to 21.9 m and are constructed of reinforced concrete, taking the form of heavy main beams supporting secondary ribs 230 mm wide at 1.37 m spacings. The structure was designed so that the same steel shutters could be used for varying rib depths. Variations to depths were made with steel skirts added to the bottom shutter pans.

The architectural design of the building called for large volumes of floor space in the concourse areas uninterrupted by columns and consequently a large proportion of the mezzanine floors are hung from the main floor structure above. These mezzanine floors span approximately 11 m in both directions and are designed as 460 mm thick two-way coffered slabs with ribs 190 mm wide at 0.9 m centres.

A special system of propping the floors during the construction stages had to be evolved because of the unusually tall floor to floor heights, the large spans and the hanging mezzanines. These props were positioned on a 2.74 m grid to suit the module of the building and remained in position during the course of construction until the upper portions of the structure were self-supporting.

25 mm wide expansion joints running east–west and north–south through the complete height of the building split the structure into six separate sections and control the expansion and contraction of the building.

Approximately 23,000 m³ of concrete were placed in the international terminal building during construction and 3,000 tons of reinforcing steel were used. Because of the large spans involved, special rollings of steel had to be ordered.

The domestic arrivals concourse consists basically of a basement, ground floor and roof slab with a first floor slab forming a balcony to a large double volume central area. The structural frame is reinforced concrete again with spans

Fig. 2
Detail of strip timber panelling in TV auditorium

Fig. 3 above
Detail of in-transit passenger bedrooms on roof of terminal complex

Fig. 4 below
Detail of overhead pedestrian link between administrative office block and terminal building constructed with structural steel

in the order of 11 m. The floors are supported by 690 mm deep main beams and 530 mm deep secondary beams spaced at 1.37 m centres. The roof to the concourse forms an interesting structural component and was constructed as a two-way coffered slab spanning 24.4 m and extending for 45.7 m with ribs at 2.74 m spacings in the east–west direction and 1.37 m spacings in the north–south direction.

Elevated motorway to the terminal buildings

The motorway on the land side of the terminal buildings consists of a four-level, reinforced concrete structure comprising the basement, a ground floor roadway, first floor roadway with roof above. The upper roadway serves international departing passengers and the lower roadway the remaining passenger movements.

The Porte Cochère portion of the roadway spans 20.1 m and is supported every 5.5 m by the architecturally conceived grille wall which is used structurally to carry the loads down to the foundation level.

The upper roadway has approach and exit ramps 9.1 m wide which take the form of a bridge-type structure supported on single central columns at 16.5 m spacings. The portion of motorway in front of the domestic arrivals and departures concourses is designed to allow for future remodelling of the old concourse. The structure consists of precast, prestressed, trough-shaped beams spanning 20.1 m onto in situ, reinforced concrete, portal frames with columns at 16.5 m spacings.

The complete motorway complex absorbed 20,600 m^3 of concrete and 3,000 tons of rein-

forcing steel in its construction and is approximately 640 m long from the beginning of the approach ramp to the end of the exit ramp.

Administration building

The administration building consists of three basement levels and eight floors above ground. The building relies upon the central core for its wind stability and the floors are generally of solid flat slab construction. The building is supported on piles and is designed as an integral part of the parking garage which surrounds it.

Underground parking garage

The excavation of the two-storey, underground parking garage was started in January 1970. The garage consists of two parking levels below ground with a brick-paved piazza and ornamental fountain on the roof.

The lower floor of the parking garage is below natural water level, which necessitated a sophisticated under-floor gravity drainage system. The drainage system was installed and put into operation during excavation, thus enabling the contractor to work on a dry site.

The basic grid of the garage is a 17.1 m by 2.54 m module. The garage has been so designed that extension can take place in the future without disrupting the operation of the garage and the existing drainage, electrical and sprinkler services.

Access roads

The road system is designed to cater for traffic arriving from Johannesburg, Pretoria, Kempton Park and the East Rand via the provincial road interchange to the west of the airport building complex. The traffic is directed either to the upper motorway for departing international passengers or to the ground level motorway which serves arriving international passengers and all domestic flights. An alternative route makes it possible to proceed directly to the parking areas both underground and at surface level. Provision has been made for the future when domestic passengers will depart from the upper level motorway as well. The exit roads lead traffic from the terminal buildings and the parking areas back into the provincial interchange.

The road system serving the new terminal buildings also integrates with the roads serving the new freight areas to the north of the terminal. The whole internal airport road system virtually forms an interchange in its own right and the roads interlink through a system of under- and overpasses. The largest of the three bridges incorporated in the scheme is a prestressed concrete bridge with a central span of 29.3 m.

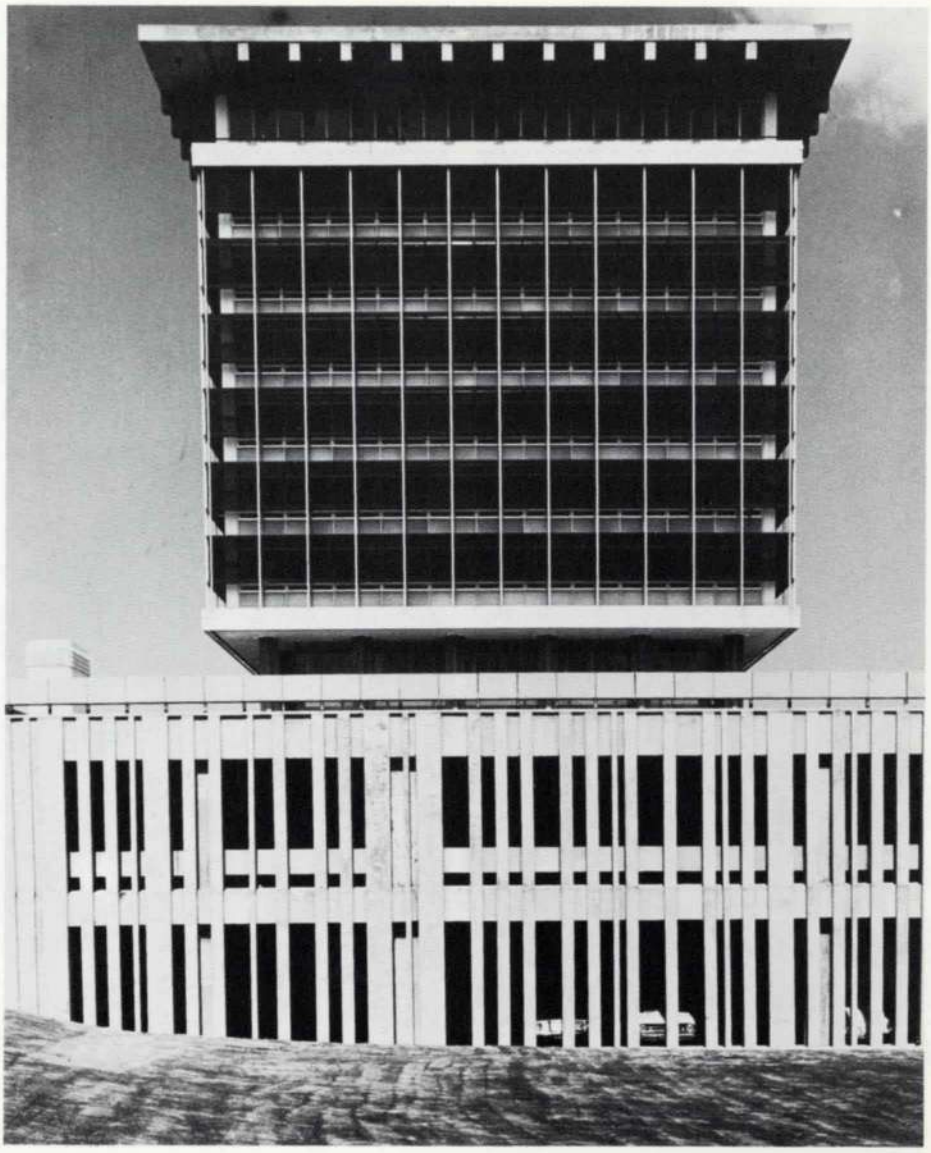

Fig. 5
Elevational view of administration office block with double level underground parking garage in the foreground

Fig. 6
The public section of the first floor departure concourses. Note the granite finish to columns

The roads in the terminal area extend over a length of 7 km, whilst the link road and internal service roads to the new freight complex will be approximately 5 km long. The open-air parking in the terminal precinct covers approximately 6 hectares.

A considerable proportion of the roads is in deep cut, and founding conditions at this level, at which poor clayey material and the natural water table were encountered, presented problems both from a drainage and strength point of view. These roads required a depth of construction of 0.9 m with excessive subsurface drainage in the lower areas.

It was obvious that the construction of these roads and other services in the area would have to be very carefully staged and programmed to fit in with the construction programme of the new terminal buildings and the new underground parking garage. Access to and from the airport had to be maintained at all times and many deviations were necessary for all the new roads to be built through existing routes and parking areas.

As existing parking areas were drastically reduced to allow new construction to proceed, new parking areas had to be made available. Full account had to be taken of the Provincial Administration's programme of construction for its highway and interchange outside the airport, which also included extensive deviation works, and the airport roads programme had to be phased in with this work. A detailed procedure chart thus formed an essential part of the tender and contract documents for the airport roads system. This predetermined programming was a major feat of engineering planning and was highly successful in ensuring uninterrupted flow of traffic throughout the construction period.

Water supply and fire protection

Water for domestic and fire fighting purposes at the passenger terminal and freight complexes is stored in a 6.8 million litre reservoir from which it is pumped through 300 mm and 350 mm steel ring mains. These mains are bitumen-lined and wrapped with bitumen-impregnated fibre glass, and joints have a moulded bitumen casing. The pipes are further protected against electrolytic corrosion by means of an impressed current cathodic system integrated with the same system which is used for the fuel supply line under the apron.

The domestic water for the terminal buildings is pumped to tanks situated in the service tower of the main building and in the roof of the office block by two two-stage centrifugal pumps, the operation of which is automatically controlled by float switches in the tanks.

Water for fire fighting purposes is taken to specially designed fire hydrants on the apron and on the land side of the main buildings. These hydrants have a calculated flow of 6,800 l/min per hydrant unit. Fire hose reels in the buildings and sprinkler systems in the parking garage and in the basements of the main buildings are also fed from the fire main. The freight complex will have similar provisions.

Three diesel-operated, single-stage, centrifugal pumps are provided in the reservoir pumphouse for boosting the water pressure in the ring main for fire-fighting purposes. Operation of these diesel pumps is controlled by flow switches in the hydrants and sprinkler systems.

A central control panel in the main building indicates operation of any of the flow switches and provides manual control of the pumps.

Stormwater and sewage drains

The stormwater drainage system at the airport discharges under the apron and runway system towards a low lying area to the east of the main runway.

Most of the extensive subsurface and surface drainage to the new terminal buildings, the underground parking garage, the new roads and surface car parks, are drained to these outlets to avoid cutting through the existing runways. Stormwater runoffs have however increased considerably with the build-up of surfaced areas at the airport and additional outlets to the west, through the new provincial road system, have been used to assist with the disposal of stormwater.

Sewage from the terminal buildings and the new hotel on the airport property is conveyed through a twin 230 mm sewer running under the terminal buildings and out to the north directly into the Kempton Park system. A separate 375 mm diameter connection takes the sewage from the new freight buildings which also house the new flight kitchens.

Conclusion

At the time of the inaugural SAA Jumbo Jet flight to London on Friday, 10 December 1971, more than three-quarters of the terminal buildings had been handed over to the Department of Transport as scheduled, together with phase one of the roads contract and upper parking level. The completed building complex is due for final handover during the second half of this year followed shortly afterwards by phase two of the roads contracts including three bridges. The final handover of the roads contract will have to phase in with the completion of the provincial road's interchange.

However, work is now proceeding on the two new freight complexes at Jan Smuts, one for SAA and the other for foreign carriers. The firm is involved as consulting civil and structural engineers on both these projects and once again the brief includes structures, road design and traffic control, sewage and stormwater disposal and reticulation for water supply.

Fig. 7
View on to apron from public area of first floor departure concourse. Note steel cruciform columns supporting roof in the background

Fig. 8
View of first floor departure concourse. Entrance to the concourse is from the motorway on the right and check-in counters are on the left

Credits

Clients: SA Public Works Department
Architects: Erik Todd, Austin, Sandilands and Partners
Quantity Surveyors: Quantity Surveying Consortium
(Photos: Lynda Shave)

Finite element computations of ground movements beneath a trial embankment

Brian Simpson

This paper won second place in the competition for the 1972 Cooling Prize of the British Geotechnical Society.

Introduction

In the design of large embankments for roads and dams, and of structures for which foundation movements are of critical importance, it is increasingly necessary to obtain accurate predictions of displacements. For this purpose the soil mechanics group at Cambridge University has proposed several stress–strain models of soil behaviour, and in the last three years I have developed a computer program which uses these models to predict displacements in soil masses by means of finite element techniques.

The program has been used successfully to predict the results of model tests in the laboratory, but it was desirable also to test it against full-scale field trials. An opportunity for this arose when data from a trial embankment for the Kings Lynn Bypass were kindly made available by Mr P. F. Wilkes, Project Engineer to the Norfolk County Council. The project has been described in detail by Wilkes[7].

The site

The Kings Lynn Bypass will skirt the town to the south and will consist largely of embankment built on fenland. The typical site chosen for the trial was about 750 m east of the Great Ouse; it had previously been used for a bowling green. The trial was conducted with the two aims of testing the effectiveness of sand drains and finding the maximum height to which an embankment could be built rapidly. On the plan of the embankment in Fig. 1, the four instrumented cross-sections are indicated; two of these (Sections 3 and 4) had sand drains and one of the undrained sections (Section 2) was loaded rapidly to failure. The embankment was heavily instrumented, and only those instruments which will be referred to later are shown in Fig. 1. The existing road embankment to the south of the trial probably affected the failure mode of Section 2.

A site investigation was carried out by Messrs Soil Mechanics Ltd. The strata found for Section 2 are shown in Fig. 2a, and similar strata were found for the other sections.

The stress–strain model

In the finite element method, a set of constitutive equations is required to relate the stresses and strains of each element so that overall equilibrium and compatibility conditions can be applied. A mathematical model relating strains to effective stress was developed after studying the findings of a number of workers at Cambridge University on the deformation of remoulded kaolin. In its main features, however, the model is consistent with the known characteristics of many soils and uses as its most significant parameters the angle of internal friction, $\emptyset'$, the compression index, C_c, and the parameter Γ. (If a normally consolidated soil fails in shear at voids ratio e_f and mean normal stress p_f, then $\Gamma = e_f + C_c \log_{10} p_f$). The model may be regarded as a further development of the 'Cam-clay' model[3], and is intended to be suitable for a wider variety of soils and soil states. In the Appendix, 'Cam-clay' is briefly defined and the modifications are listed.

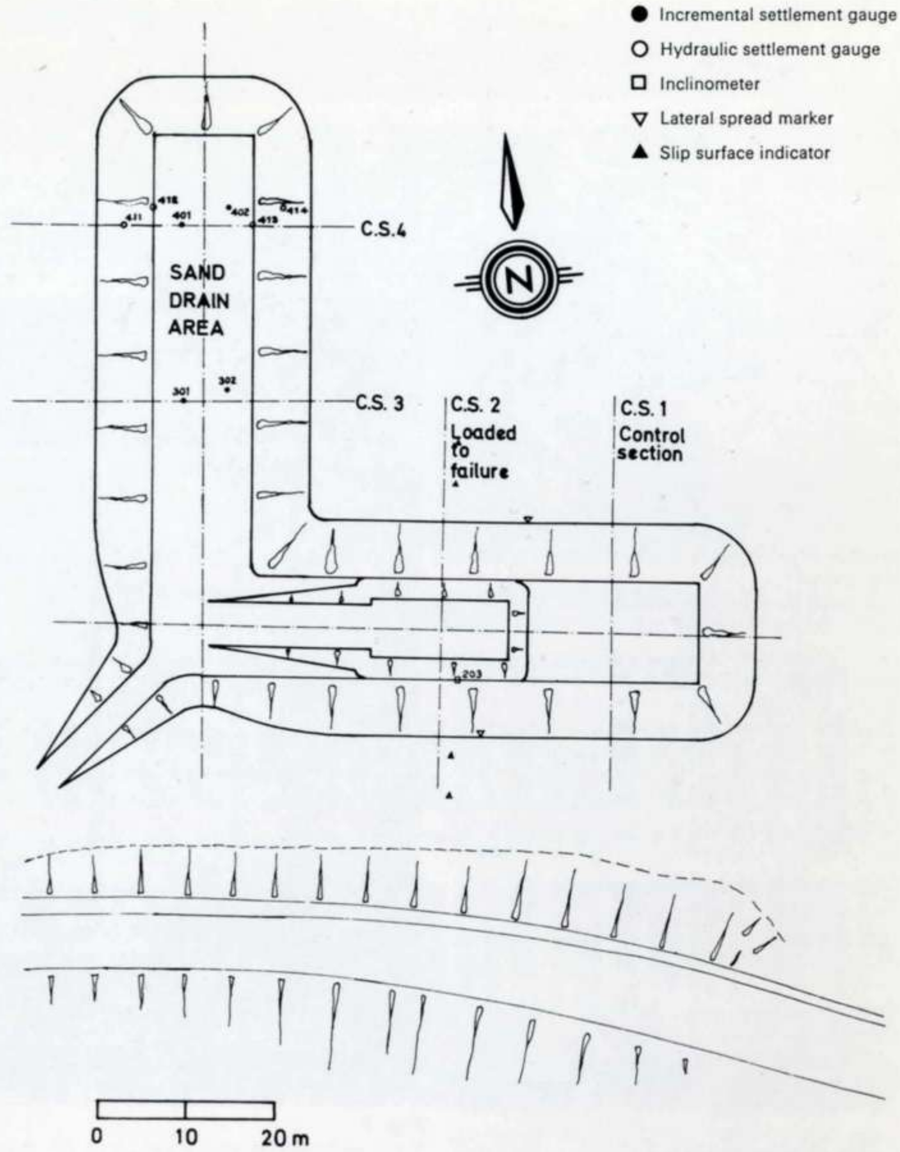

Fig. 1
Plan of the trial embankment

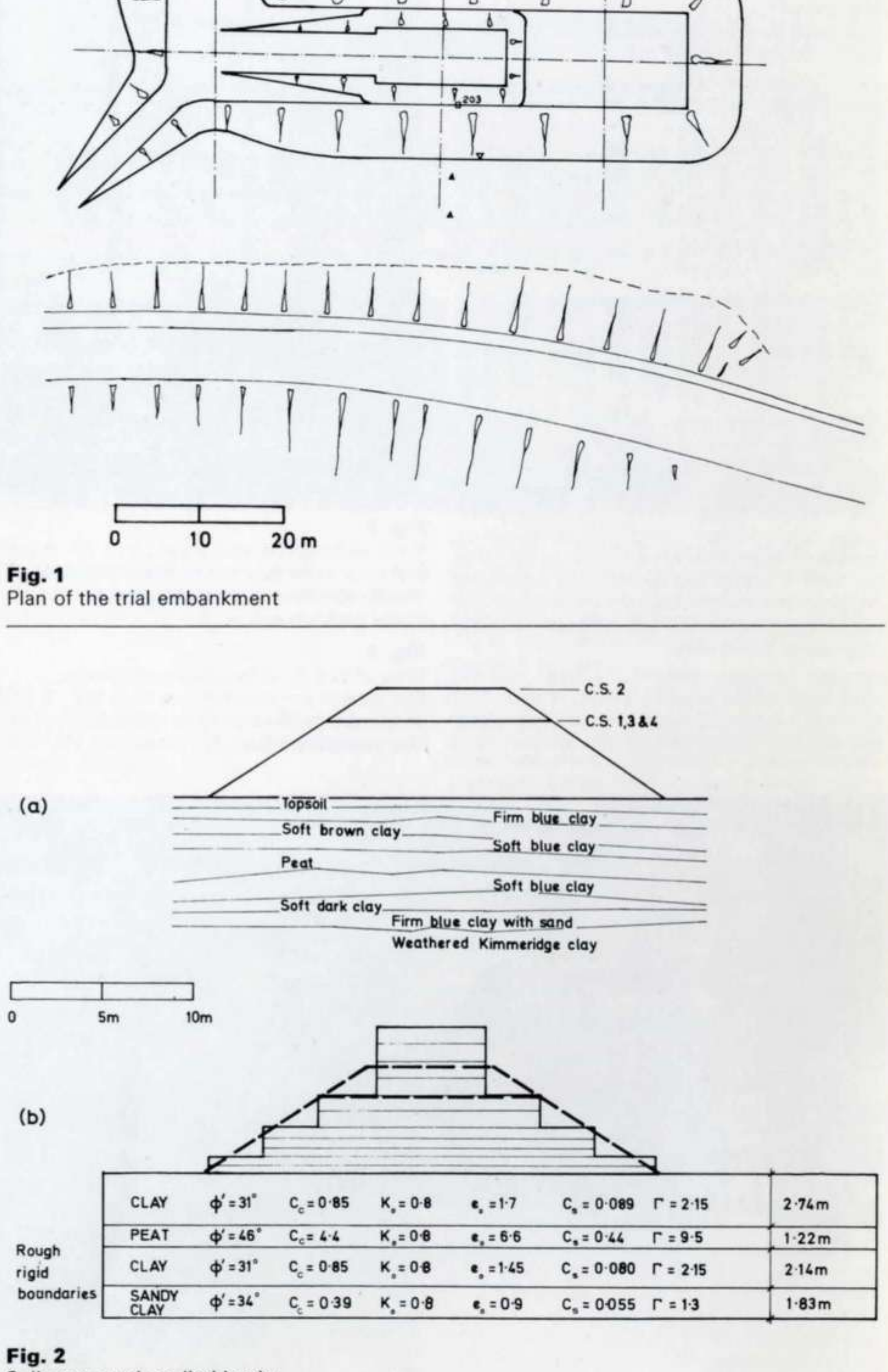

Fig. 2
Soil strata and applied loads:
(a) in the trial (Section 2),
(b) as assumed for computational purposes

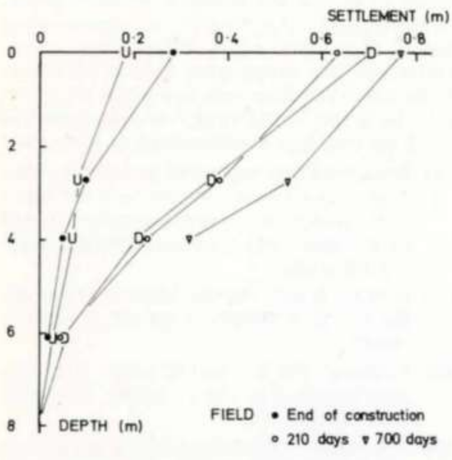

Fig. 3
Settlement profiles for Sections 3 and 4 (averaged) at 5.75m from the centre line

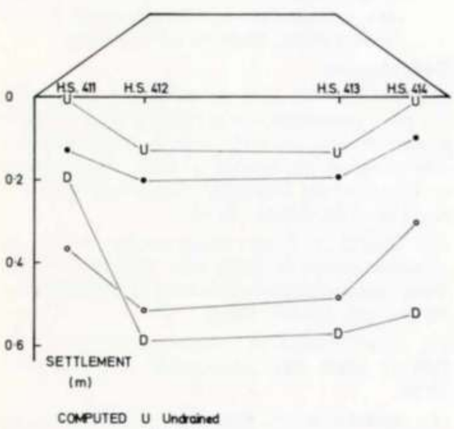

Fig. 4
Settlements at ground level for Section 4: readings taken from hydraulic settlement gauges 411-414

The computations

Since the measured soil parameters showed considerable scatter, it was convenient to idealize the ground conditions to give the four strata with properties as shown in Fig. 2b. The main parameters required were all derived directly by averaging values given in the site investigation report. Values of K_o and initial voids ratio were chosen to correspond to a very lightly overconsolidated material, and other parameters, which have been found to be of minor importance[5] were taken from experience or literature related to similar soils.

In the computations the embankment was represented by a vertical load acting on the soil surface. Fig. 2b shows how the load was applied incrementally, being spread uniformly over the whole area of the embankment in the first increment, but with later increments gradually concentrated nearer to the centre line.

The computations were designed to model the behaviour of Section 2, and for this reason the lateral boundary taken for the soil supporting the embankment is closer at one side, representing the influence of the existing road embankment mentioned above.

Two computations were performed, one representing fully undrained behaviour (the short term response to rapid loading) and the second representing fully drained behaviour (the response at the end of primary consolidation); the same stress–strain model and parameters were used for both computations. For the drained case, the water table was taken to be at a constant level of 1.25 m below ground level; this depth was found in the field trial to be 1.0 m on average.

It was assumed that the soil was 99% saturated and, in the undrained case, erratic fluctuations in predicted pore pressures between adjacent elements were smoothed out between increments of load. This process is believed to represent satisfactorily the behaviour in the ground[5].

In the drained and undrained cases, 156 and 200 constant strain triangular elements were used respectively. Although these were sufficient to represent the behaviour of the ground at moderate displacements, many more elements would be required to study failure adequately.

Results

I had access to data from the instruments as shown in Fig. 1. For Sections 3 and 4, which had sand drains in the trial, displacements at the end of construction (which took about 20 days) and after further periods of 210 and, where known, 700 days, will be compared with the predictions for the undrained and fully drained conditions. Further settlement after 700 days is thought to be due mainly to secondary consolidation. For Section 2, which was tested to failure, attention will be concentrated on conditions shortly before failure.

Settlements

To give the most useful comparison, the results from incremental settlement gauges 301, 302, 401 and 402, which were all equidistant from the centre line (Fig. 1), have been averaged; the standard deviation was found to be about 5% of the largest settlement. The settlements, measured at the changes of strata, are compared with the predicted values in Fig. 3. It is seen that both short term and long term settlements are successfully predicted.

Settlements at ground level were measured by hydraulic settlement gauges. The computed results and those measured for Section 4 are shown in Fig. 4. It is seen that the short term and longer term (210 days) field measurements are bracketed by the computed settlements. The effect of the asymmetric boundary conditions is clearly shown in the computed results for the drained case.

Horizontal movements

In Fig. 5 horizontal displacements at 5.75 m from the embankment centre line are shown for two stages of loading shortly before failure. The field measurements, taken from Section 2 (Inclinometer 203 – see Fig. 1) which was loaded rapidly and had no sand drains, are compared against predictions for the undrained case. It is seen that the agreement is again good.

Fig. 6 shows the increase of measured lateral spread of Section 2 as the embankment was built up. Also shown are the computed curves for the drained and the undrained cases. It is notable that the measured lateral spread was close to the computed curve for the drained case in the early stages and moved towards the undrained curve in the later stages, for which the construction rate was higher.

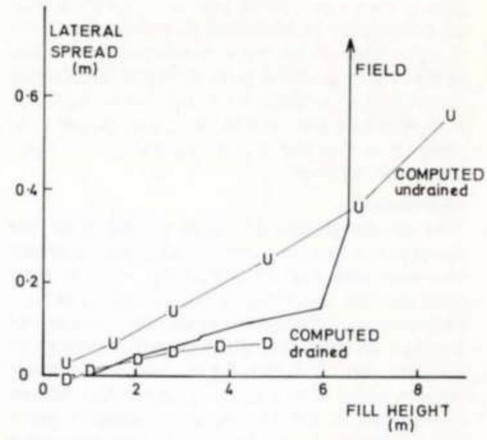

Fig. 5
Horizontal displacement profiles for Section 2 at 5.75m from the centre line

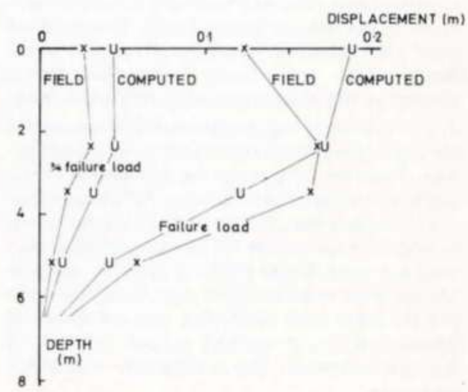

Fig. 6
Increase of lateral spread as embankment constructed

Pore pressures

The computed excess pore pressures show less agreement with field values than do the displacements. Fig. 7 shows the contours of computed excess pore pressures compared with

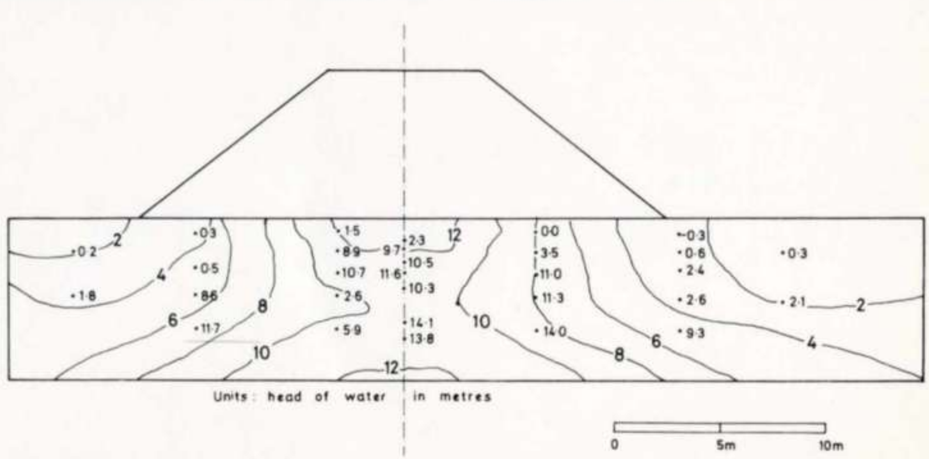

Fig. 7
Computed and measured excess pore pressures for Section 2 just before failure

piezometer readings for Section 2, recorded just before failure. The predictions allow for no drainage, even at the ground level, where there was a drainage layer in the trial, and so it is to be expected that predicted pore pressures near the surface will be high. Predicted pore pressures at depth are, in general, lower than those measured. The reason for this is not known, but it is felt that the results are good enough to show that the techniques adopted hold considerable promise.

Discussion

The results presented have shown that the programme can be used to predict successfulm the two extremes of behaviour – short terly (undrained) and long term (drained). Vertical settlements could probably be predicted amost as well by the one-dimensional methods in current use, but the finite element method allows good predictions of horizontal movements also. To achieve this in a problem involving a soft clay taken to large strains requires a complicated model of soil behaviour. The results presented here, when considered together with equally accurate predictions for laboratory model tests, suggest that the stress–strain model used is satisfactory. The main parameters required were obtained directly from a standard site investigation report. Previous workers have assumed isotropic elastic behaviour[2] or an elastic stress field[1]. The method used here, however, employs stress and strain fields which are entirely compatible, being related by the elastic-plastic constitutive laws.

The influence of the lateral boundaries selected for the computation requires further investigation. From an early stage the movement of the soil follows the pattern which is fully developed at failure; it is therefore probable that the lateral boundaries have little influence, provided that they are outside the limits of the slip surface. Movements much smaller than those beneath the fill were both predicted and measured at heave and thrust markers outside the toe of the embankment; the correlation was poor, however.

The rapidly loaded section of the trial embankment failed along a narrow zone. This could not be modelled by the mesh of finite elements used here. Simpson and Wroth[5] describe the effectiveness of a method of refining the mesh automatically in regions of high strain and so modelling the formation of a narrow slip zone.

Secondary consolidation will be considerable in the organic clays and is not predicted by the present program. However, other workers in the field of finite elements have successfully modelled problems involving creep behaviour[8].

Conclusions

The finite element method has the potential to provide the link between academic theories of stress–strain behaviour and the field predictions required by practising engineers. In this paper results produced using a complicated model of soil behaviour based on parameters derived from a commercial site investigation have been presented and the agreement with field measurements has been shown to be close.

This type of theoretical work can never reduce the importance of field tests, but the computations would cost at commercial rates only a small fraction of the cost of the embankment trial and could be repeated for varying soil conditions and soil parameters at neighbouring sites.

Acknowledgements

I wish to thank Mr W. H. Spencer, Director, Eastern Road Construction Unit, and Mr P. Deavin, County Surveyor, Norfolk County Council, for permission to present this paper. I am particularly grateful to Mr P. F. Wilkes for having made the relevant field data available, and to Dr C. P. Wroth, my research supervisor, for much help and encouragement. The computations were carried out on the Titan computer which is part of the facilities of the Cambridge University Computer Laboratory.

Appendix

The stress–strain model

Cam-clay represents an ideal elastic-plastic material, limited to axi-symmetric conditions of stress and strain. Fig. 8 shows how the plot of stresses is divided into two regions by a yield locus. Within the yield locus only volumetric elastic strain occurs; it is assumed that there is no elastic shearing. If the stresses in a small region of the material change, so that the stress state crosses the yield surface in the plot of Fig. 8, plastic deformation takes place accompanied by work-hardening; the yield surface expands and the amount of expansion determines the plastic shear strain. The yield locus is assumed to be a plastic potential and a flow rule is derived. The yield surface is shaped so that the ratio of shear to volumetric plastic strains becomes very large at a ratio of principa

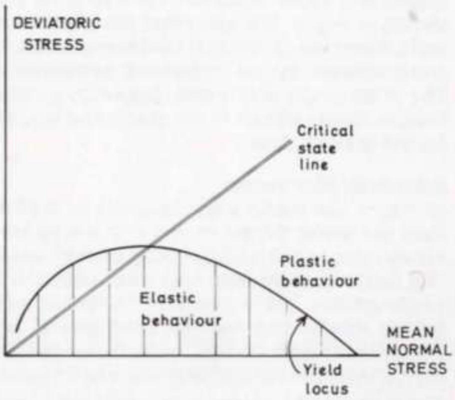

Fig. 8
The yield locus as used in the stress-strain model

stresses which is a material constant called the critical state stress ratio. (This is related to the value of ϕ' for the material when normally consolidated). Deformation at stress ratios higher than this causes dilation and work-softening. The model used for the computations described here was similar to that outlined by Simpson and Wroth[5] and incorporated the following four modifications to Cam-clay.

(a) The model was expressed in terms appropriate to plane strain. In particular the mean of the two principal stresses in the plane of deformation was substituted for the mean normal stress.

(b) Elastic shear strains were introduced, assuming a constant elastic Poisson's ratio[6].

(c) Although the concept of a flow rule was maintained, the rule of plastic normality was disregarded.

(d) The curve in stress space which was the yield locus of the Cam-clay model was employed in a different way. Deformation and work-hardening accompanied the expansion of the locus, but, for changes of stress within the locus, plastic strains occurred whenever the stress state moved towards the locus, and the plastic stiffness was determined by the distance of the current stress state inside the locus.

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Design and construction innovations: systems utilizing panel components

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Introduction

Man's basic requirements for shelter are a roof, walls, and preferably a floor. He has provided these in a variety of ways according to the climate, the materials available and the area in which he lives. As his skill in use of tools has improved so has his degree of sophistication in the assembly of the materials, which in its turn has led to an increase in size and amenity of the buildings that he has constructed.

The era of industrialization with its emphasis on the use of machines and mass production techniques to make consumer goods available to all but the very poor, has somehow not resolved the problem of housing in the urban environments that it has created.

The wider spectrum of the problems involved have been discussed in depth by others at this conference and possible solutions have been argued. On the assumption that the socio-economic problems have been resolved we, the designers, are still given the same basic criteria, i.e. using today's materials and today's tools we must provide the shelter needed today.

It is my brief to discuss the use of one of those materials, in the various large panel systems that have been utilized so widely in the construction of housing and other associated buildings.

History

The use of concrete as a building material has really only gained recognition since the turn of the century. It comes as a surprise, therefore, to realize that as early as 1905, in Liverpool, a man called Brodie was casting room size wall, floor and ceiling panels in a central depot and hauling them by steam tractor to be erected on site. A three-storey apartment building still exists. From the '20s through the '40s various attempts were made to introduce techniques which were not too dissimilar from those used today, but although numbers of successful buildings were completed both here and in Europe, the large scale development of these ideas did not take place.

It was not until the '50s that circumstances in Europe created a favourable climate for the large scale usage of these systems. These circumstances are all well known. The aftermath of two world wars created a tremendous and urgent demand to replace houses destroyed and slums that would no longer be tolerated by the emerging societies. There was an acute shortage of traditional building materials and skilled labour was at a premium. Builders became more receptive to new plant-orientated techniques and suppliers were able to revive the dormant ideas and to suggest new ones. In France and Eastern Europe the governments were able to ensure a continuous programme of work which enabled development to take place with an assured return on the heavy capital investment required. Great Britain was relatively late in the field but in 1968 produced 21,000 of the 700,000 homes built using large panel systems in Europe that year.

Why conrete panels?

It is interesting to note that of the 22 successful bids under the Break-through programme, (see Appendix), nine systems propose the use of concrete as the main structural material and, of these, seven are load-bearing panel systems. In the light of European experience this was not unexpected.

In the first place, concrete itself is a relatively cheap building material, it is durable, lends itself to a variety of architectural treatments and has readily available sources of raw material which, with a minimal amount of pre-treatment, can be incorporated in the building.

Given sufficient repetition, substantial savings in cost and improvements in quality can be achieved by precasting components, in either central or on-site plants. The most usual demonstration of this is in the large scale production of precast flooring of various proprietary forms.

Traditionally, multi-storey construction evolves around the erection of a structural skeleton, the bones of which are beams and columns, either in steel or concrete. Floors are added and this forms the frame to which space dividers, external façade treatment and services are added to suit the particular needs of the client or his architect.

Several systems have been developed based on a continuation of this approach and, in situations demanding maximum degrees of flexibility in planning, they are beginning to demonstrate their worth. There are certain design and construction pitfalls, notably in the areas of connections and tolerances.

Within the apartment sector the extreme flexibility is an unnecessary luxury and other, more demanding, criteria must be met. In addition to structural strength, particularly in high rise buildings, there is the need for minimum sound transference, for adequate fire resistance between dwelling units, for low maintenance costs and, if the national problem is to be met, for speed of construction.

Systems incorporating the use of large load-bearing concrete panels supporting concrete floors to which they are sensibly jointed, will fulfil all of these requirements and at the same time make the least inroads into the already scarce skilled building trades. In erecting the structure not only are the bones provided but also a substantial amount of the flesh, which will eliminate follow-up trades.

Production methods

Despite the differing methods of production and assembly, all the systems can be recognized by the types of component produced.

Load-bearing walls are normally 150 or 180 mm thick, dimensions which will provide the necessary sound insulation between dwellings and at the same time provide structural strength for buildings of approximately 20 storeys without the provision of special reinforcement. They are usually cast vertically in multicell steel batteries. This provides a smooth face to both sides of the wall which then requires the minimum amount of preparation to receive a painted or papered finish. Some systems manufacture to standard modular lengths, usually 1.2 m or 2.4 m; others attempt to provide room size units based on a more flexible module.

Most producers incorporate electrical conduit with switch and socket outlets; this may be pre-wired but it is not usual. Other plumbing and mechanical installations can be cast into the wall provided that they can be accommodated in the thickness. More often holes and chases are provided to receive sub-assemblies at a later date. Openings are also provided for doors and windows with the frames frequently cast in.

The greatest difference between the systems occurs in the production of floor panels. These may be cored or solid, prestressed, post-tensioned or normally reinforced. The method will often depend on the background of the sponsor.

Specialists in precast plank will tend to design around their product which provides pre-stressed panels of up to 12 m span in 0.6 m or 1.2 m widths. Considerations of differential deflections, horizontal joints and surface finish have to be carefully taken into account.

Others utilize horizontal forms, carefully engineered to provide normally reinforced units of high dimensional accuracy. The bottom surface produces a smooth mould face, the top is trowelled and covered by a suitable finish. The trowelling can be by hand or mechanical and the applied finish may be incorporated at the time of manufacture. By placing the moulds on a production line, they can be moved to operational stations, including those of concreting and curing, which provides a continuous cycle of operations. The heavy capitalization involved in this type of plant demands the maximum amount of repetition of a few number of variations in slab types.

Floor slabs can also be cast in vertical batteries in a method similar to the production of walls. The advantage of obtaining two smooth surfaces is an obvious one. The equipment occupies less floor space and the same labour crews can produce both floors and walls. The size of slabs can be varied within the confines of the battery cells. Careful quality control is essential and to maintain economic utilization of the equipment requires production planning of a high calibre.

There are other less common techniques which have now passed the experimental stages. One produces panels by pouring a wet concrete mix into a mould and squeezing the excess water out under high pressure. It is possible to produce four panels per hour from one mould using this method. Unless a completely standardized product is desired, the rate of production could be an embarrassment. Further development work should resolve this anomaly.

All production methods allow for the formation of holes and sleeves to accommodate plumbing and mechanical services.

The architectural cladding can range from the rather brutal plain concrete panels with rectangular windows offering the only relief, to very heavily profiled units using multi-coloured aggregates and cements or mosaic and ceramic finishes. The former is very cheap but has created a public reaction against 'low cost precast housing'. The other extreme can be visually more satisfying but is generally achieved with some sacrifice of the internal amenities because of its greater cost.

Curtain walling, plastics, brickwork and other proprietary panels have all been used in an effort to achieve variety.

Storage and transportation

It is tempting to think only of production and erection processes, since these are the areas where the most dramatic demonstrations of technology are to be seen. Unfortunately, there have been a large number of builders lured by the temptation to dismiss half their labour force and produce apartments at an astonishing rate merely by purchasing Brand-X equipment.

Without the superior management skills which are the key to the exercise, the misuse of the plant can lead to inevitable bankruptcy. One mistake per week made in the traditional manner can be more easily discovered and rectified than the same mistake repeated in 50 panels before it is realized on site. The higher the production rate the more vulnerable is the system.

Careful attention must be paid to the balance between production and assembly. A plant producing 40 dwellings per week will soon be forced to curtail production if the erection rate drops to 35 dwellings. There must be adequate storage space to allow for unpredictable weather or labour difficulties. This storage must be planned to avoid double handling of units, otherwise the cranes will spend more time searching and sorting than they will loading panels. The stacking frames must be well designed to avoid the consequences of accidental damage when moving units weighing 10 tons or more.

There are always certain critical panels which fall outside of the main production run. In the event that these are not available when required then erection must cease despite an abundance of general stock. Although these may seem to be elementary points, lack of attention to them can soon create a situation where the increased productive capacity will only serve to multiply the problems.

It is in this field that the training of the professional can be deployed to advantage, but he must be aware of the complete process in order that his designs can be economically produced.

Erection

Erection is usually carried out by teams of eight to 10 men serving a crane. The type and capacity of crane is a function of the system philosophy. Smaller units can be handled by light mobile cranes which can collect their own panels from loaded trailers. Large room size panels require less number of lifts per floor but need heavier cranes with sensitive slow speed controls to assist the crews in locating the panels.

In Europe these requirements have resulted in the design and manufacture of specialist families of tower and portal cranes which provide the facilities needed by a system sponsor. Once again the designer must be aware of the capabilities of the equipment.

Where other prepackaged subsystems are employed it is usual to place them in position prior to the next level of floor slabs being erected.

Before the structural joints are completed the wall panels are temporarily braced whilst they are levelled and plumbed. The exact method and equipment to be used should be decided during the design phase.

The planning of the erection sequence should always consider the follow-up operations that are necessary to complete the finished building. Merely producing a concrete structure at an accelerated rate is only addressing something of the order of 35% of the total process. The building must be made weathertight as close behind the erection level as possible in order that the finishing trades can perform. These trades and subsystems must themselves be related to the total process.

Jointing

Whenever system builders, or their designers, assemble, the discussion will inevitably come around to one of two topics.

1 How do you tie the panels together?

2 How do you keep the weather out?

The first of these has tended to attract the greatest publicity since the accident at Ronan Point, and quite rightly structural safety is of paramount importance. Unfortunately, there has been an infinitely higher failure rate in the second field due to lack of attention to small details at the junction of units.

The basic principles of structural connections are now fairly well established. The details of implementation constitute another major area of difference between the systems. The essential tying together can be achieved mechanically by welded or bolted devices or by means of 'wet' reinforced or post-tensioned concrete techniques.

The mechanical solutions call for a high degree of accuracy in casting and assembly; they also involve the use of skilled tradesmen to ensure a sound connection. Provision has to be made for temporary stability in order that valuable crane time is not absorbed during erection.

Most Western European systems tend to favour structural joints using cast-in-place concrete and dry packed mortars. The use and disposal of reinforcement in the joints depends on the method of manufacture and the sequence of erection. These operations are more tolerant of dimensional inaccuracies, can be carried out independently of the placing of units and require no particular high skill. Normal cold weather precautions have to be adopted during winter but the early enclosure of the building space helps to minimize exposure to the elements.

Weatherproofing can once again be classified by family type. From the designers' viewpoint it is tempting to call for plain-ended units and joints caulked with mastic. The quality of mastics has improved enormously over the years and, provided that manufacturing and erection tolerances are maintained with high quality workmanship in their application, a sound joint will ensue. It is rarely possible to execute this work as erection proceeds. As an alternative, preformed compressible strips may be inserted into grooves cast in the panels, but variation in joint widths can affect their efficiency.

Open-drained joints are most often used for a variety of reasons. They can accommodate variation in joint width and thermal movement. Since they are made at the time of erection, their formation and inspection can take place from inside the building. The materials used are not dependent on age or weathering characteristics and consequently the joint is maintenance free. It does require good detailing, however, particularly at the intersection of vertical and horizontal joints and can create manufacturing difficulties for elaborately profiled architectural panels. It has been thoroughly tested in exposed coastal conditions in Norway and Scotland and rarely gives trouble.

Thermal insulation

Where concrete is employed as the cladding material, thermal insulation is normally achieved through the use of sandwich panels. Difficulties can arise at the joint between panels and around openings where there may be a discontinuity in the insulation giving rise to thermal bridges. The relative impermeability of concrete surfaces can lead to problems of condensation under combinations of low temperature and high humidity. The insulation should thus be considered in conjunction with the heating and ventilating systems.

Electrical and mechanical services

It has not yet been found practical to achieve the ideal situation in which all the sub-systems can be pre-assembled, placed in position and merely connected up. Even where the components are available, their final location can interfere with the temporary propping necessary until the joints are structurally sound.

An economic storey height will not allow for long horizontal ducts with complicated intersections. The tendency is to provide a series of vertical service cores which contain utilities for dwellings in the immediate vicinity.

Prefabricated plumbing assemblies are used and can work, but care is needed in the consideration of tolerances and movement. Since pipe connections are normally located in relatively restricted areas, it is often difficult to join one unwieldy set of components to those already fixed. The requirements of some codes can also inhibit prefabrication.

Accurate precasting will provide holes for pre-planned independent arrangements which can be fixed efficiently. To allow for flexibility a combination of holes can be provided and those not used filled in afterwards.

The manufacture and use of complete kitchen and bathroom assemblies in off-site factories is not a technical problem. Unless a large market is assured, however, it is still at this time cheaper to use rationalized traditional methods. Safeguards must also be provided to protect these finished products during construction.

Heating and ventilating systems are normally from traditional sources. Once again they are supplied vertically, unless individual units are used, and distributed horizontally within the dwelling. A convenient location is next to the façade panels which can be profiled to accommodate pipes or fans. Some wall or floor panels do have heating elements cast in. Elevator shaft walls incorporate guides and inserts which allow for completely pre-assembled cars and motors to be installed prior to putting the roof on. This single factor can reduce the completion time of a 20-storey building by three months.

Although electrical distribution is provided through cast-in conduit, this technique is not without problems. A series of walls produced from the same mould may have a number of electrical variations which are not always apparent to the man producing or placing the panels. Horizontal distribution is usually through pre-formed baseboards incorporating raceways for electricity, telephone and television cables.

Economics and disciplines

The main aim of this presentation has been to describe what constitutes a large panel system, how it functions, and how it relates to the other subsystems which form the finished building. This methodology is fine provided that it is not just considered to be an exercise in technical virtuosity. It must be able to demonstrate that these buildings can either be obtained more quickly or more cheaply or preferably both. In Europe this has been so, particularly in the medium to high rise field. In the United States it is probably still too early to draw any valid conclusions. Traditional costs themselves are tremendously variable and can range from as low as \$13/ft^2 to \$34/ft^2 in the New York area. Such predictions as we have been able to make suggest that, with a properly planned and controlled operation, savings of the order of 7% can readily be achieved but that, with the present generation of systems, it is doubtful if figures of more than 10 to 11% are going to be exceeded even in the most efficient conditions. There are, however, unseen savings to be realized in early completions on site. Being able to hand over a building with a 30% time saving, for example, can lower the construction loan interest by 3%. At the same time, income from the building is realized at an earlier date. In areas of urban renewal the cost advantages which accrue from time saving are possibly not as important as providing new homes for those in need.

To achieve these savings there are some basic requirements, the first of which must be good design. There is no such thing as an infinitely flexible system and the designer must recognize the disciplines and constraints with which he has to work. Having recognized them, they need not necessarily be considered to be unduly restrictive. The worst possible exercise is to take an existing design and try to convert it to a system solution.

Having realized a good design there must be a continuing market. The capital cost of setting up the equipment for even the most unsophisticated system is as high as \$1 m and can go up to the \$5m mark. A continuous demand over a number of years can be very much more valuable than one large order requiring heavy capitalization in equipment with no follow-up activities to maintain the plant utilization. Some factories can exist on 300 or 400 units a year, others need 1,000-2,000.

Since the material content is the same, it is clear that the major economies are to be made in the more efficient use of labour and in creating opportunities for relatively unskilled labour. This can only be successful with the cooperation of the unions. It is not only a question of basic wage levels but also of numbers of persons employed per operation. If a normal erection crew of eight to 10 men has to be increased to 20 or more because of demarcation disputes and trade practices, the system is defeated.

It is also very important to have good management at all levels. It is not enough to have top level people in production and construction alone. A key to any successful system is the design which it must meet. The reason why large panel systems can be so competitive is because the problems have been thought out beforehand. This requires a complete collaboration on the part of the architect, the engineer, and all the various mechanical and electrical consultants and subcontractors. The information that they can supply is needed at the very outset in order that moulds can be set up, equipment ordered and production schedules arranged. This must mean the abandonment of the concept of the isolated professional working in his own sphere with little concern for the problems of others engaged in the building process. The leadership of this new design team will largely be governed by the organization sponsoring the system, but can be architect, engineer, contractor, or client. The omission of any of the participants at the start can only lead to an out-of-balance system. If the concept of industrialized building only serves to get us all to operate in our own jobs in a more efficient way, then this alone would justify its introduction.

Appendix: Operation Breakthrough

Operation Breakthrough was the name given by the Department of Housing and Urban Development to an ambitious programme launched with the object of linking the space age technology to the solution of the acute housing and urban renewal problems of the USA.

An international competition was created in late 1969 which invited 'innovative' proposals based on planning, technical, financial and development skills. This aroused the interest of the whole of American industry and most of the European system sponsors.

22 prototype sites across the USA were to be developed and the 10 most successful systems were to be given top priority to the 'set-aside' funds provided by the Government over the next 10 years. The target was 26 million homes.

This provided the incentive for Wates Ltd. of London to form a joint venture with the Rouse Company of Maryland (the 6th largest mortgage bankers in the USA). Arups were invited to provide engineering support in North America.

The Rouse-Wates submission was amongst the initial 22 selected and successfully completed a scheme of 240 apartments in St. Louis. They were also amongst the three or four final survivors of the various political, financial and bureaucratic battles that eliminated some 400 of the original hopeful contenders.

The uncompleted final phase of Operation Breakthrough, the 'set-aside' funds, was abandoned together with all Federally assisted housing programmes in January 1973 and an 18 month moratorium on these programmes has been ordered by the new Nixon administration.

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