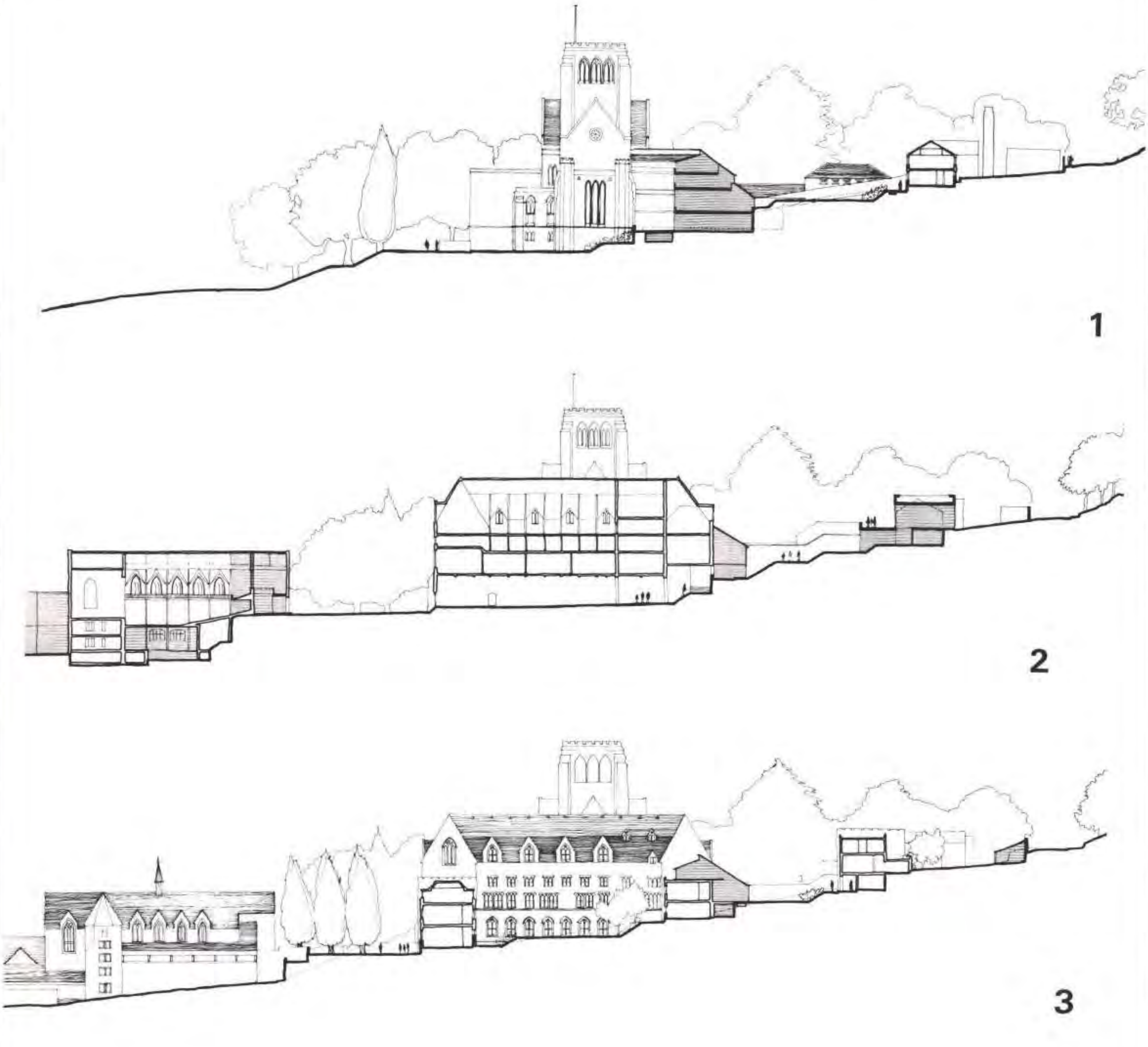
THE ARUP JOURNAL

DECEMBER 1975



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New buildings for Ampleforth

Richard Frewer

Ampleforth is a Benedictine community, with a public school for some 800 boys, set in the midst of the Yorkshire countryside. The community came to the village of Ampleforth in 1802, when it occupied a single country house looking south from a wooded slope over a wide valley. In this setting of outstanding natural beauty the community flourished and grew.

Most of the building around the house originates from the late Victorian era, and is in that favourite Victorian style – baronial ecclesiastical Gothic. The largest part of the work was carried through by the Gilbert Scotts; Sir George (1811–1878) designed the monastery and main school block; his grandson, Sir Giles (1880–1960) designed the Abbey (finished in the late 1950s, and one of the finest examples of Elizabeth II Gothic), the classroom quadrangle and the refectory. He also planned further extensions which were never executed; a clock-tower on the south-east corner of the school buildings and a school gate-house to the west of the school refectory. (Figs 1 and 2).

A wide walkway passage links all these buildings and this, together with the buildings themselves, was an influential factor in our design. The natural environment was also influential. From anywhere on the Ampleforth site, the view over the valley is dominant. The

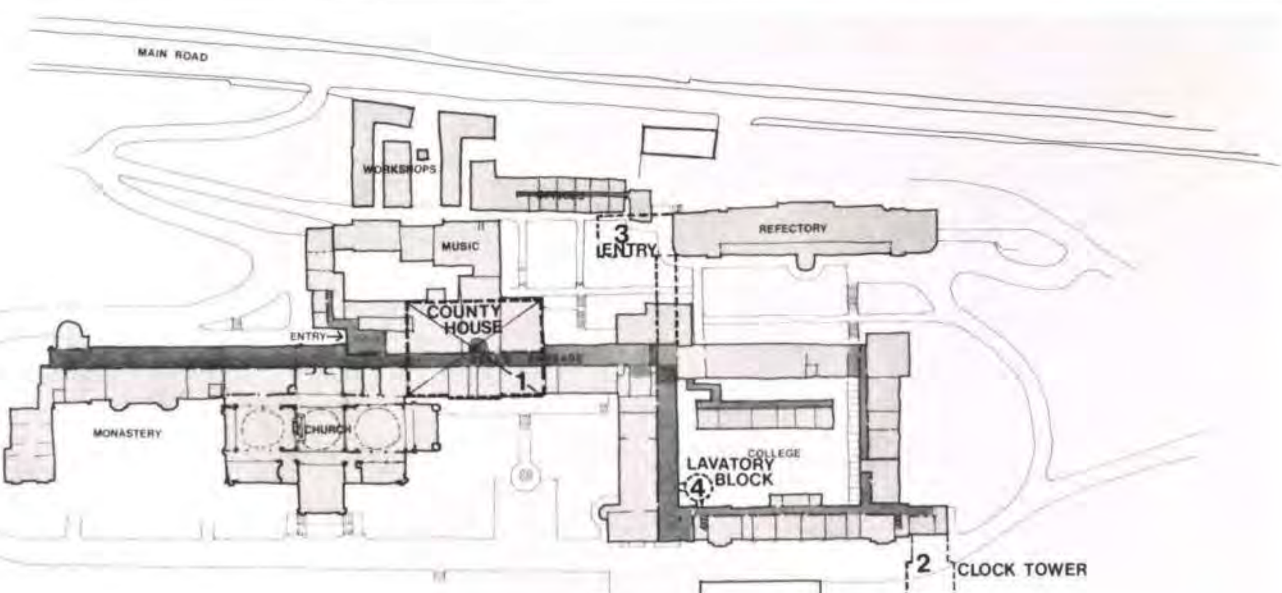

Fig. 1
1968 existing plan

Key
1 Original country house
2 Proposed clock tower
3 Proposed entry
4 Proposed lavatory block

valley changes hourly, and has an enormously powerful influence on the community. The windows face south over the view, and the building, with its weight and sculptural strength, makes a cliff-like formation on the side of the valley. This is the essence of Ampleforth's architecture.

When the community first approached Arup Associates in 1968, they wanted a number of small additions to the school quickly; lavatories and bathrooms, study carrels, and temporary classrooms. At the same time they wanted us to look at long-term possibilities for extending the school and monastery buildings together.

The 800 boys in the school are divided up into houses, each containing about 50 boys. As the first houses were built in the 19th century, you can imagine how primitive some of them are. Our first job was to build the new lavatories and bathrooms for two of these houses. In examining this project, however, it became clear that, rather than embark on new buildings, the problem could be better and more cheaply handled by freeing certain areas in existing school buildings, and changing their uses.

At this stage it was agreed that we should prepare a feasibility study which would combine a thorough examination of education policy by the community itself, with a study by us of possible new building, and re-use of existing space.

Feasibility study

The full feasibility study was presented in 1970. In addition to the original brief, it examined the service problems of the school (e.g. the solid fuel boilers installed c.1930, as was the boilerman.)

The major changes proposed were as follows:

(1) Demolition of the central building as it was so shaky. The double house which it contained was to be rebuilt with the same organization as the present houses. However, it was to be adaptable to give more single study/bedrooms. The community saw the possibility of being forced into becoming a sixth form college if education policy continued as it was.

(2) Provision of a central dining system and new kitchens to ease staffing problems

(3) Provision of a sixth-form house in the centre of the site between monastery and school to give the older boys peace for study and to encourage closer links between them and the monastery

(4) Rationalization of car and pedestrian circulation

(5) Provision of elaborate sports facilities

(6) Provision of classroom areas and workshops

(7) Renovation and change of a lot of existing space.

The main design ideas to emerge in the feasibility study were:

(1) The cliff-like formation of the building must be retained and emphasized.

(2) The design of the roof is very important, as all buildings are seen from above.

(3) All materials used should appear weighty.

(4) The central walkway should be extended outwards.

(5) The east-west line of the buildings, if added to, and the central buildings which are part of that line, should form a strong spine to which all other buildings should be attached (like a string of beads).

(6) At any stage the incomplete plan must appear complete.

(7) The preservation of old trees and the planting of new ones are of great importance.

(8) The standard to which we build must be compatible with the standard of the existing building. (We set the expenditure between state school and University Grants Committee levels) (see Figs. 3 and 4).

Fig. 2
Model of plan 1968

Fig. 3
Model of feasibility plan 1970

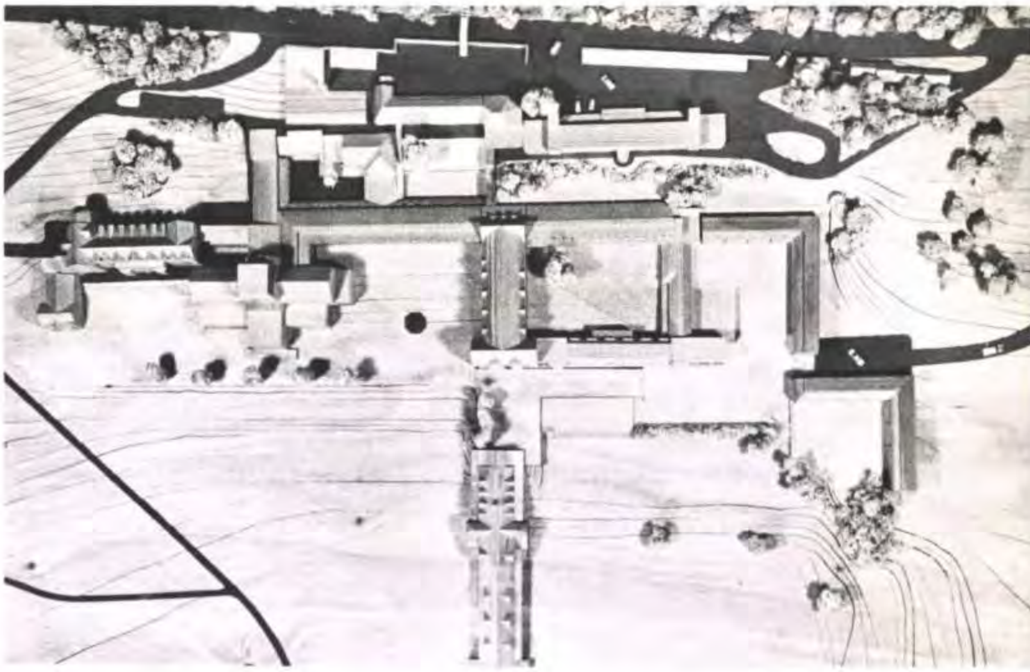

Fig. 4
Model of feasibility plan developed

Key

A Classroom block, Phase 1
A2 Classroom block
B Double house, Phase 1
C Central building
D Workshops and sixth form rooms
E Sports complex
F Entry building
G Services centre and workshops

New buildings for Ampleforth

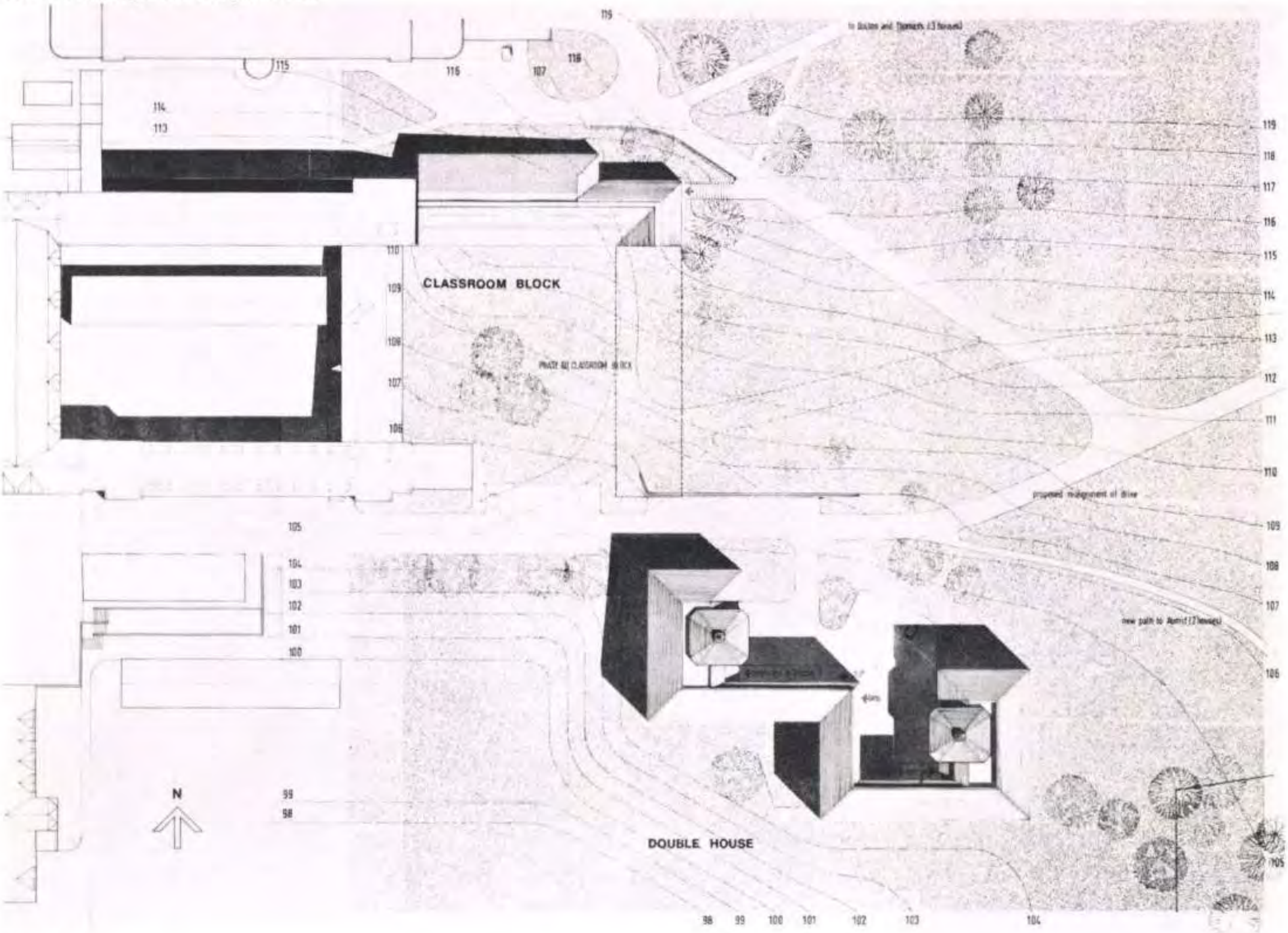

Fig. 5
Phase 1, site plan

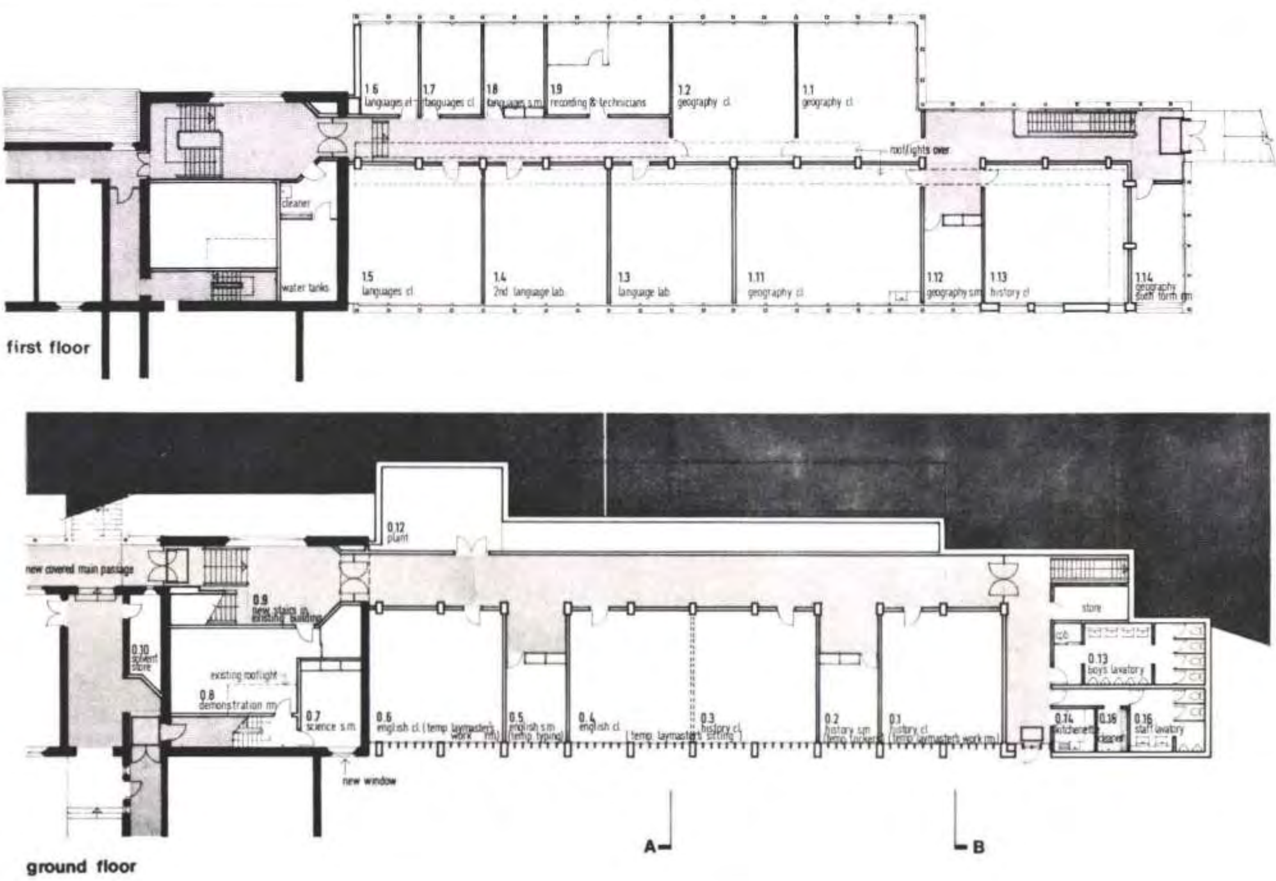

Fig. 6
Classroom plans

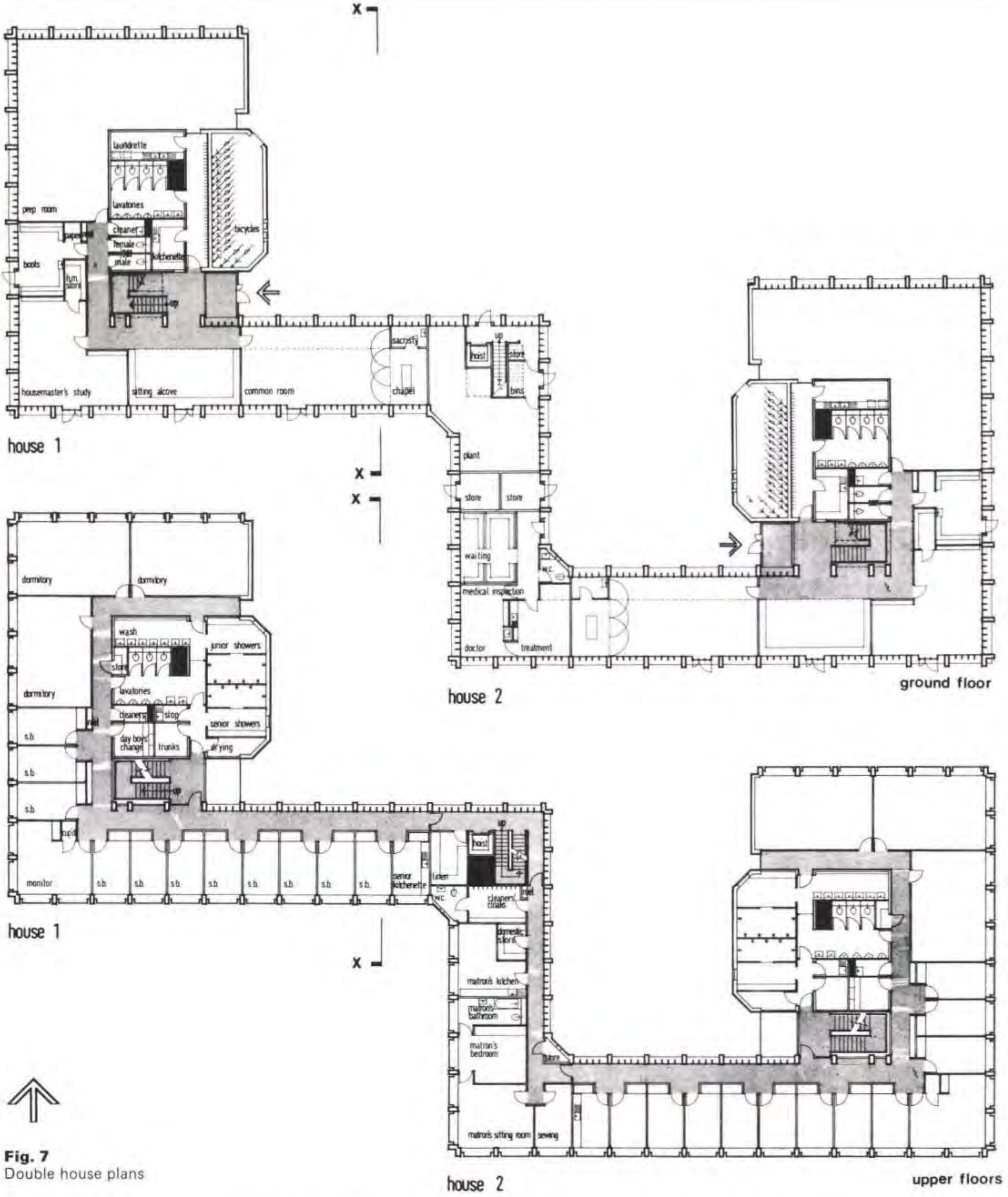

Fig. 7
Double house plans

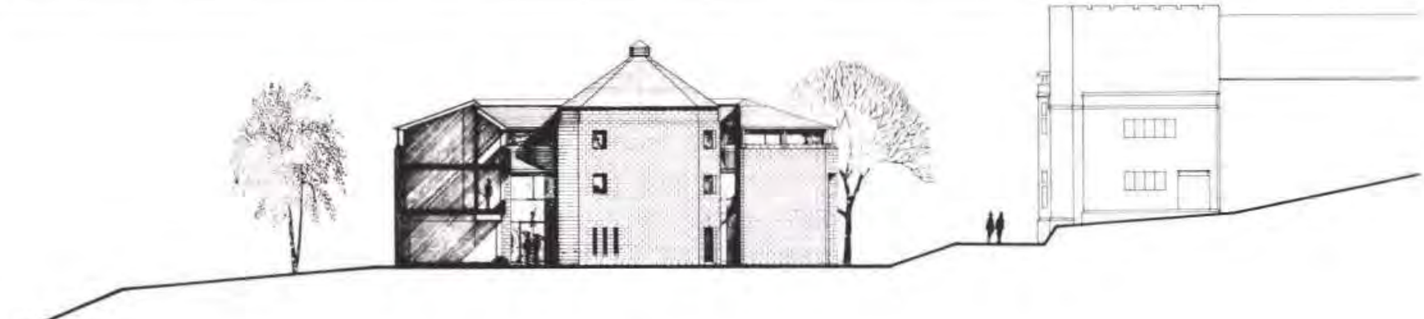

Fig. 8
Double house section

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Phase I

This feasibility study was accepted and we were asked to continue into the first phase. This comprised the double boarding house and the classroom block with consequent conversions. The idea was to clear the central building of essentials, knock it down and rebuild it. We would then provide a new house for senior boys, central dining facilities related to a new refectory for the monastery, and numerous lecture rooms to form a new centre-piece to the school.

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Choice of building materials

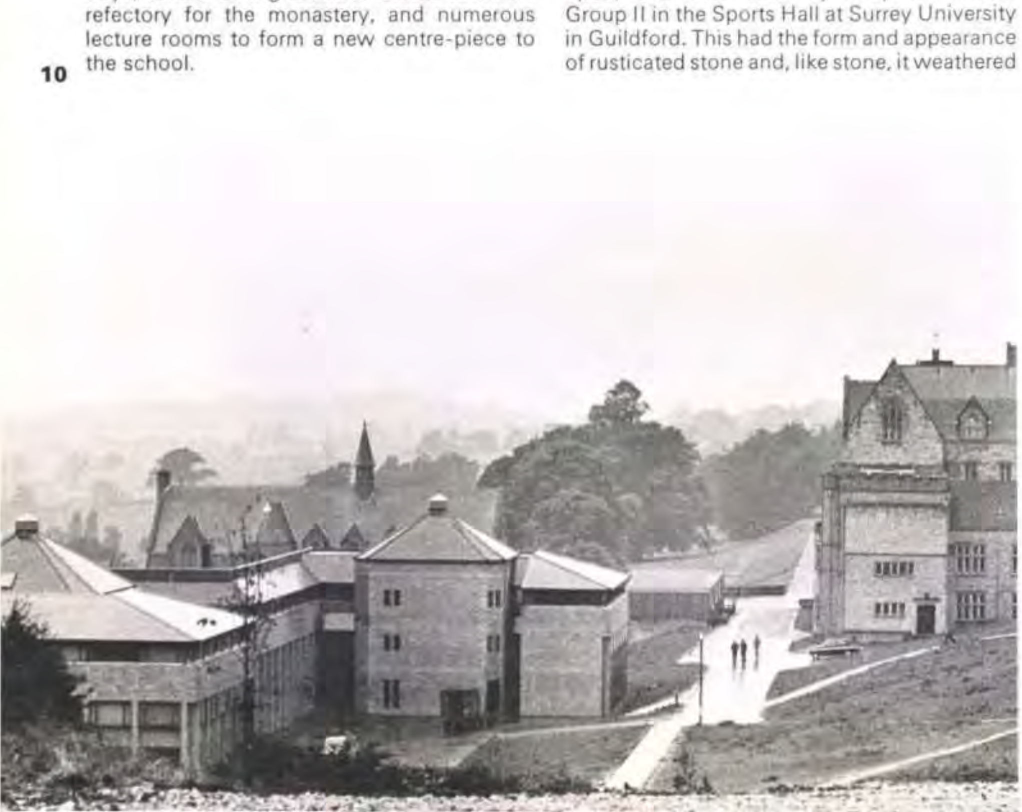
In developing the ideas from the feasibility study into buildings, one of the major problems was the choice of facing materials. Stone seemed the right answer but was an economic impossibility. However, as we have already stated, a material with visual strength was essential. A concrete block had been developed, and first used by Arup Associates Group II in the Sports Hall at Surrey University in Guildford. This had the form and appearance of rusticated stone and, like stone, it weathered well; so experiments were done with different stone aggregates to get as near as possible to the colour of Ampleforth. This sort of match is an extremely difficult one to make, and, although samples were erected and tested on site, we didn't know with any certainty what it would look like until it was complete. Now the buildings are finished, we can see that the colour blends, and that the apparent weight and scale of the block are as close as we could have wished to the stone.

Building anatomy

The two new buildings have a similar anatomy. Both construction and service methods are repeated, although the width of the bays, which governs the planning, differs. In each case the roof of slate sits as a floating plane over the blockwork. This has the effect of stressing the strength of the walls and adding a lightness to the roof. The blockwork piers, which march regularly round the building, are deep. This means that although the solid surface area is small, the building appears, in perspective, to be very solid. The windows recess, to give a feeling of deep gap, and in places they are brought to the surface to give reflectivity and a liveliness to the buildings' personality.

Interiors

The interiors at Ampleforth are rich in Thompson oak furniture and panelling. Thompson was a local craftsman who was encouraged by the monastery. All his work for them was in oak and is signed with a little mouse carved into the furniture. This oak furniture gives the school a most unusual added quality. We could not afford quality hardwood, but were concerned to reduce the number of materials used to a minimum, as the constant use of materials



helps to produce clarity. Consequently, all the woodwork was stained dark, and was treated as a rough element in the buildings. Against this background the furniture, much of which is in Thompson's hand (and the rest traditional), looks very elegant, but substantial enough for the tough quality of the interiors.

The study bedrooms

The senior 24 boys of each house have their own study-bedrooms. These, traditionally, are the most basic little boxes – but out of them the boys express their own individuality. Some rooms become temples to the God Mao, others become expressions of right wing nationalism.

Fig. 9
South face of school with double house

Fig. 10
View of the valley over double house

Fig. 11
The blockwork ties up
with the grain of the existing stone

11

12

13

Fig. 12
Study bedroom top floor

Fig. 13
Study bedroom middle floor

We were able to produce rooms with variety by the section we used. Some rooms are in the sloping roof and have rough, tough wood ceilings; some are flat ceilinged and lower. We have provided means of pinning things to all free wall space and a large window which in all cases relates to the main view.

The future

Phase I is complete. Of the future, who is to say? But we have confidence that whatever is built on the central site, the language of architecture which has been developed for Ampleforth has 'rightness' or 'truth' and is sufficiently adaptable to remain relevant.

Architects + engineers + quantity surveyors:
Arup Associates Group 3.

Main contractor:
John Laing & Son Ltd.

Fig. 14
Classroom block from double house

Fig. 15
The bay

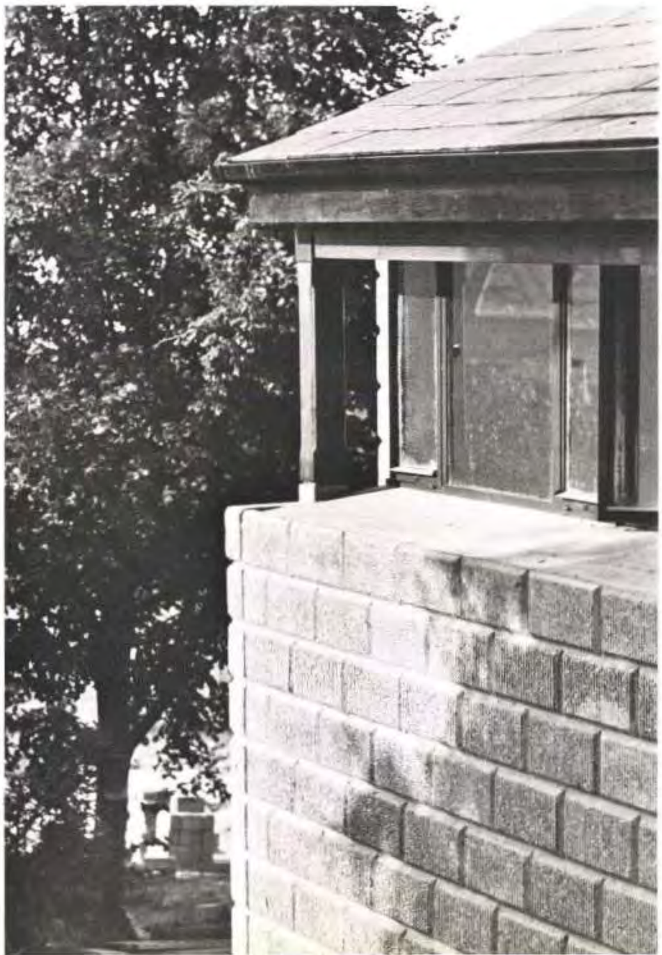


Fig. 16
Double house entrance

Fig. 17
Corner detail

Photos:
Arup Associates

Wind engineering: the personal view of a practising engineer

Ken Anthony

This paper was presented at the 4th International Conference on Wind Effects on Buildings and Structures, ,Heathrow Hotel, 8–12 September 1975.

Introduction

It was about 100 years ago that it really started, with the loss of many lives in the Tay Bridge disaster. Interest and activity in the field of wind effects on and around buildings and structures has continued more or less ever since. It was in the late 1950's that the subject as we recognize it today really got under way. That was the time when the vast early warning radar systems were being designed and built and when Davenport was winding himself up to make such an impression on the scene as his contribution has clearly made.

Now, here we are in 1975 at the end of the conference, the fourth in a series started in 1963. How long ago that seems! Apart from these of the international kind, there have been innumerable other conferences, symposia and meetings around the world on similar and related topics. The numbers of papers offered to conferences increases year by year, one might almost say 'expotentially'. This conference alone received over 140 offers, of which, of necessity, only a third could be accepted for presentation. The total number of papers and articles published over the last 15 years or so now runs into thousands, representing years of research and investigation by scores of teams and groups in many countries. One only has to read current references and bibliography lists to realize how interest and active work on wind loading and effects has undergone a 'population explosion' of its own. The British Hydromechanics Research Association Abstracts list about 5000. Even if one maintains an interest and involvement in the subject, it is virtually impossible to keep abreast of the rapidly expanding literature, especially if one is a practising engineer or architect who has, to say the least, other professional interests and duties.

What progress?

A fundamental question arising out of this is – 'How closer is the practising engineer or architect to being able to solve his day-to-day windy problems or, by design, to avoid them?' I would expect that if this question were put to those of us to whom the subject is some sort of speciality, the answer would be that certainly progress has been made, especially in the last decade, but that the progress is not justified by the vast amount of effort put in. To cite just one example, I came across a reference recently to a paper reviewing the last 60 years of research into the Karman vortex street! Similarly, at almost every conference one finds papers on recurring specific subjects such as wind-induced oscillations of circular cylinders and suspension bridge decks. One wonders how much more there can be to find out about such phenomena of practical use in design. Given the vast amount of data available on the whole subject of wind loading and effects, the man in practice with special knowledge has the advantage of being able to sift out the relevant information he needs but more often than not there is something about it which precludes anything like direct application. Very often also, the data are conflicting or at least there are significant discrepancies between the results of various workers. This leads to time-consuming and frustrating comparisons and adjustments of data, at the end of which engineering judgment has to be exercised. It is right and proper that judgment should be exercised but it is unsettling sometimes to have to rest one's responsibilities on some of the data one has to use.

But what of the typical engineer (if you will excuse the expression)? How does he cope? Generally speaking his problems fall into two main categories – overall wind loading for structural stability and surface pressures for the design of elements of cladding. One would not expect him to have to plough through the esoteric literature to find answers to these problems. He needs readily-available concise applicable data. In the UK, he will immediately turn to *CP3*, Chapter V, Part 2. Some informed engineers will disagree, but in my opinion, today's *CP3* is a vast improvement on its previous version and now that it has been complemented by Eaton and Newberry's *Wind Manual*, it has become an even more valuable document. But it is no panacea for the day-to-day design office problems.

Another very valuable compendium of easily-applied information is the External Flow series of data items prepared and published by the Engineering Sciences Data Unit (ESDU). These are produced from virtually every known source of data and include well-explained introductions to the subject. They are less well-known than they deserve to be. But neither *CP3* nor ESDU data items are yet able to go far enough to help the engineer assess either overall loadings or surface pressures and their distributions on anything but the most simple form shapes under limited conditions of turbulence. This latter aspect is becoming increasingly relevant as consulting engineers are being asked to stipulate glazing loads, despite the fact that they may have no responsibility in this respect. The situation is not helped by the lack of understanding by architects and glass manufacturers of the random nature of wind loads, the philosophy of acceptable risk and the ensuing probability statistics. As soon as the typical engineer's problems step outside the most simple, advice has to be sought elsewhere, often experts in the field outside the engineer's own organization. I should like to say a few words about this later but before doing so and in order to be fair about the impression I may have given so far, it may be useful to review the status of some of the aspects of wind loading and effects of most relevance to the practising engineer and architect.

Some areas of common office interest

Prediction of design wind speeds

As all loads and pressures derive from appropriate design wind speeds, the prediction of the latter is of fundamental importance. This is well appreciated by all countries in which the built environment has been, and is being, developed. It is becoming so in those countries which intend (rightly or wrongly) to follow suit and embark on dense development. The result is that appropriate raw wind data are being collected, recorded and statistically processed in a manner of direct use to the design engineer. Various 'established' statistical models have been adopted and generally accepted. But there are certain dangers to be aware of when using data from some parts of the world. Apart from the most obvious questions of the averaging interval of wind speed being recorded, one sometimes finds that the data have a mixed population. Such an example is Malaysia where windspeeds are generally low but are punctuated by high gust speeds deriving from thunderstorm squalls. The relationships between, say, mean hourly and gust speeds in such instances are not similar to those applying to winds of cyclonic origin.

One might have said that provided one is aware of such phenomena, the statistics of wind speed prediction, established by such workers as Gumbel, Thom and others, was a fairly quiet subject. But now, at this conference, Simiu and Filliben have thrown a pebble into this millpond by concluding from their extensive analyses that this may well not be the case, especially in hurricane and typhoon-prone zones. The results of their future proposed work is looked forward to with anticipation.

Overall loading and response of tall, sharp-edged buildings

The tall building which stands well out of its surrounding terrain without proximity effects is not too difficult to analyze nowadays for loading and response provided it is of simple plan form and elevation for which appropriate drag coefficients are readily available. By this is meant coefficients appropriate to the turbulence to be experienced. Other provisos are that the structural damping and the terrain roughness can be reasonably accurately estimated. These two parameters remain problematic because of their considerable influence on the response. Still, too little is known about the damping of full-scale superstructures, let alone the effects of their various foundation systems. There is a wide range of values to choose from but the right choice is critical. Likewise, there exist descriptions of terrain with designated roughness lengths or friction coefficients but it is sometimes difficult to select an appropriate one, especially for sites in hilly areas. The fact that the American and Canadian codes give guidance on the loading and response of wind-sensitive structures is an indication that research work has filtered down to grass roots level but the application still has limitations. On the other hand, the British code merely recognizes the problem.

It is popularly supposed that sharp-edged or so-called bluff bodies are Reynold's Number independent, but it is also known that if flow reattachment occurs, then Reynolds casts his spell once more. This phenomenon is of significance at higher intensities of turbulence or when the plan shape of the building has a long dimension parallel to the wind compared with its cross-dimension. But then lots of our buildings are of this slab form and most experience such turbulence. Again, it can occur with wind directions slightly inclined to the major faces and, as we know, this case is inevitable. This latter case can also produce sizable torsions on the building arising out of asymmetrical pressure distributions. These are areas not yet adequately covered in the form of design guidance of practical day-to-day office use.

The problems of wind loading assessment for tall buildings becomes particularly acute in the office as soon as the shape or form becomes 'unusual' or when existing or future proximity effects become of concern. As most tall buildings are 'one off' designs for city or urban sites, these two factors are commonplace. There is no generally applicable guidance on them because of their particular project nature. In such cases, resort has to be made usually to wind tunnel tests of one sort or another. Although wind tunnel facilities are being extended and improved, the time taken to get the investigation set up and carried out and the ensuing cost are still problems to the busy engineer, the architect and the client. It will probably be some time before the client can be persuaded that, in terms of value compared with the total building costs, a wind tunnel test, when necessary, is on a par with the soils investigation, the necessity of which he takes, nowadays, for granted.

One can foresee the proximity question becoming a matter of litigation for loading and environmental wind effects.

Wind loading of low rise buildings

There is a general impression, I believe, that the wind loading of low rise buildings is much less of a problem than for the dramatic high rise building and consequently calls for less detailed study and provision. The fact remains, however, that the vast majority of incidences of wind damage occur with low buildings, particularly domestic ones. Perhaps this is because of the complacent attitude which prevails, especially so with the detailing of roofs and cladding and their connections. Most of the handy data on drag and surface pressure coefficients, such as those in *CP3* and the Swiss Code, to give just two examples, are derived from old wind tunnel work employing smooth flow and little or no wind shear. *CP3* does have an inbuilt allowance for the effects of turbulence on local pressures but the coefficients quoted can be well exceeded in practice. Of necessity, low buildings are situated entirely in the most turbulent region of the atmosphere. Perhaps the adoption of code values is also a contributory cause of some of the damage. Darwin would seem to be one case in point. I'm sure that those who doubt the extent of damage in the UK alone would be convinced if they were to see the file kept by Keith Eaton at the Building Research Establishment.

So, more realistic values of drag and local pressures on low rise buildings are needed. The results of the field work at Aylesbury by the BRE and the wind tunnel correlations by Tom Lawson at the University of Bristol are keenly awaited.

Wind excited oscillations of circular sections

This is a hardy perennial! The literature output on this subject continues unabated. For structures such as chimneys and stacks, it is no problem to predict the critical velocity for resonant vortex shedding, provided, of course, the vibrational frequencies of the structure are predictable with reasonable accuracy. The problem is that the critical velocities are usually low – well within the probable range, and the question then is, what range of amplitudes is to be expected? It is a function of structural damping, of course, but even if this can be reasonably assessed, as is usually the case as distinct from that of a tower block, there are other factors such as mass distribution, the forcing function and aerodynamic damping which make it difficult if not impossible to calculate. One often has to take a bit of a risk and keep some strakes up one's sleeve.

This problem is even further compounded when several similar cylinders are in close proximity and interact with each other. A certain amount of good work has been done on this but it is not easily applied generally to design in the office. Hopefully, a proposed ESDU item on this phenomenon complementing existing ones on single cylinders will provide the needed tool.

Because of their larger diameters, circular tower blocks, cooling towers and such structures as circular concrete television towers, rarely suffer from resonant vortex shedding. The critical windspeed is usually well beyond the range expected to occur. However, surface pressures are of considerable interest, either for cladding design in the case of tower blocks or for the structural design of the relatively thin shell walls of the others. At design wind speeds, the Reynold's Number is up in the 10^6 or 10^7 range. Most surface pressure distributions have been derived from wind tunnel investigations at far lower Reynold's Numbers than these so there is some doubt about the pressures and indeed drag coefficients for large diameter real structures. It is not a simple solution to have a wind tunnel investigation undertaken because at such high Reynold's Numbers, a freon or compressed air tunnel is called for. These are few and far between and expensive. A further limited extension of past work into this region would be welcomed.

Human response and reaction

There are two main areas of current study into the effects of wind on people. The first concerns the indirect effects on people occupying tall buildings resulting from movements of the structure under the action of wind. This aspect is receiving increasing attention, especially in the USA where there have been several instances of real discomfort being experienced, even to the extent of buildings being evacuated, not for reasons of safety but because the occupants' comfort and, presumably, work were being severely impaired. The separate recent work of Chen and Chang is particularly useful in outlining the ranges of human reaction to motions of various frequencies, amplitudes and accelerations in occupied tall buildings.

Other investigations have been carried out in laboratory conditions. Personally, I am inclined to place less confidence in the results of these because I suspect that one's reactions to motions at say the 60th floor of a building would be rather more unnerving than to identical ones in a controlled laboratory environment. However, it is difficult to be definitive about perception and threshold limits because the matter is so subjective and conditional upon activity, expectations and acclimatization. Perhaps guidelines to acceptable ranges of motion will one day appear in codes.

The other area of expanding interest is the external environmental wind around buildings. There is no need for me to recount instances of the sometimes quite serious effects of wind flow and turbulence that have been experiended in this country alone. No doubt we have all had such experiences from time to time ourselves. This is a subject which does not lend itself to analysis. Environmental wind tunnel techniques are now well advanced for identifying potentially troublesome areas and for quantifying the expected wind conditions. It is not yet widely realized by architects how useful a design tool the wind tunnel is in this context, from the conceptual stage when the 'disposition of masses' is being decided, to the detail stage when one is wondering if local wind pressures will keep opening the entrance doors. The wind tunnel can also provide data valuable to the mechanical services engineer with respect to the expected pressure differentials across linked intake and extract systems.

However, the main area of interest is in the effects of wind on people in and around buildings at low level. The main problem lies in establishing the criteria – what is acceptable and what is not – both in wind characteristics terms and in frequency of occurrence. This is a complex and subjective matter, even more so than response to motion, as it involves even more variables. Activity, clothing, temperature, humidity, wind speed, turbulence are all relevant. Efforts are being made by a few workers in this field, notably Lawson, Hunt and Penwarden, but there are certain differences in approach and emphasis. It is desirable that generally agreed criteria be established at least for the UK so that investigations have a common basis of comparison. Because of the physiological content of the problem, this would probably need some form of contribution from the medical profession. Another need is for full scale correlations.

These are just some of the more common involvements of an engineer in practice and their current status as I see them. As already implied, my general impression is that the vast amount of research, manifesting itself in an awesome volume of literature, contains much work of value to the practising engineer but that one would expect a greater general benefit relative to the time and cost expended. Provided that he is of the informed variety, the engineer is usually able to assess the wind loading and consequent response of his project structure but it is rare that he is satisfied with the quality of the solution. If he is a typical engineer, it is doubtful that he would get even this far without advice from one source or another.

Specialist advice

Before concluding, I should like to offer a few thoughts on this aspect of advice from specialists, particularly aerodynamicists outside the engineer's own organization. It is primarily a question of relationships between the engineering and aerodynamic practitioners.

The engineer in a consultant's office has a multifunctional role and his responsibilities range across the whole spectrum of building design, analysis, construction and economics. Also, he usually has several projects running concurrently and tends, therefore, to be under considerable pressure continuously. It is common for his involvement on any one project to last several years. Essentially, he is a general practitioner and problems of wind loading only form a small part of his project activities. It is not often that he will have any specialist knowledge of the subject and even rarer that he will have had any form of aerodynamics training. Any knowledge that the informed engineer may have was probably picked up 'accidentally' or forced upon him through some project circumstance in the distant past. Of necessity, it has to be very much of a 'teach yourself' affair.

On the other hand, the aerodynamicist is much more of a specialist. Within his field, he may have a wide range of interests, but to the typical structural engineer, building aerodynamics seems to be a fairly narrow subject shrouded with mystique and full of such incomprehensible terms as cross-correlation, coherence and co-spectra.

Given that the engineer realizes he has an aerodynamic problem and sets out to seek advice he first needs to know which (witch) doctor to consult, how long he will have to spend in the waiting room, how effective the examination will be, what value he may place on the diagnosis and above all, whether there is a cure. The degree of confidence engendered is partly a matter of the specialist's bedside manner.

What the typical engineer seeks is good, solid advice, simply expressed, and in which he can have that confidence. It is true that he often leaves it too late and expects instant help but this is a consequence of the pressures he works under. It is hoped that the aerodynamicist is aware of these and responds accordingly. Although his project involvement will be short-lived compared with the engineer, the aerodynamicist's response and commitment are all-important.

He must first assess the engineer's understanding of the subject and make any necessary allowances by trying to find a common language so that the dialogue may be effective. He needs to steer the uninitiated engineer into asking the right questions and to avoid overpowering jargon as far as possible. Having identified the problems and the scope of the advice being sought, he should be open about the extent of the help he can offer, particularly if wind tunnel testing is recommended. It is recognized that, within the field of building aerodynamics, individual specialists have leanings towards different aspects and that their technical facilities have been developed accordingly. As an extreme example, if the problem is one involving very high Reynold's Numbers and a tunnel test is needed, there is little sense in pursuing the matter very far if the particular tunnel cannot be worked in required range. The aerodynamicist should also resist the temptation, albeit rare, of taking on a commission with the knowledge that the work, while providing an extension of his existing techniques, would result in less than fully applicable data for the engineer.

Together, the engineer and his adviser must define the problem and agree the scope of the advice and the investigations needed. A programme should be established with cost limits for each stage. In other words, an agreed brief has to be formulated. This should include such matters as liaison, communications, the content and format of the eventual report, the consequences of design changes during any tunnel tests and how these are to be accommodated.

Wind tunnel testing

On the question of wind tunnel testing itself, if a project gets this far, it is most likely that an informed engineer is involved. He must ensure that he has an adequate understanding of the particular techniques to be employed in instrumentation and data processing. But at the same time, being responsible for the engineering aspects of the project and knowing something of the problems of wind and model simulation, he will be concerned with the overall accuracy of the results and their application to the full scale structure. In any wind tunnel test, there arise matters of the modelling of the atmospheric boundary layer characteristics, of the anemometry and pressure measurements, of the various similarity parameters, of the data processing and of the model itself. If the test is one for dynamic response, then the problems are compounded by the aerolastic modelling and the response measurements. All these are subject to questions of simulation and accuracy. The engineer is likely to wonder what the compounded inaccuracies add up to and what confidence he can have in the results being applicable to the real structure. Such information is not often forthcoming without prompting from the engineer, yet it is fundamental to him in assessing the overall range of design parameters. Any qualifications to any part of the test procedures, measurements and conclusions must be made clear by the aerodynamicists. Some workers define individual accuracies but avoid estimating the compounded effects!

As far as reports emanating from aerodynamicists are concerned, one would make a plea that the intended recipient be kept in mind when the language, format and results presentation are being decided. Contrary to popular belief, clients are not favourably impressed by esoteric documents meaningless to them. It helps to have synopses and conclusions couched in relatively simple terms and essential that initial queries be answered. One sometimes finds also an unnecessary preoccupation with the techniques and instrumentation, even to the extent of quoting the type numbers of all the electronic black boxes.

However, it remains the engineer's responsibility to be able to interpret and communicate the results and the consequences to the client.

The wind tunnel is a valuable tool but it does not eliminate the need for engineering judgement in the application of its results. Improved confidence in its use would be generated by more positive correlations with full scale behaviour. This aspect is receiving wider attention than of late, but more needs to be done. Unfortunately, full scale investigations are very expensive and time-consuming and can only be conducted in the main by government research agencies and universities.

Aspirations

Given the amount of effort that has been, and is being, applied in the wind engineering field and the consequent volume of literature generated, one might expect the engineering and architectural professions to be well satisfied with the way progress is being made. No doubt most of the researchers feel that they should, but, seen from the other side of the fence, one cannot be so sure.

I believe the time is now right for a consolidation of past work and for a reappraisal of priorities and direction for the future. A distillation of existing data and information is required in a form appropriate to day-to-day use in the design office. The ESDU data items may be quoted as examples in this respect in that practising engineers contribute to their formulation. *CP3* is another example but it does not reflect a very high proportion of the available information on dynamic loadings and response which, after all, have been subject to so much engineering emphasis over the last several years. Hopefully, the next version will take a lead from other codes and provide engineers with the necessary guidance to deal with at least some of these recurring problems.

As for the future, one hopes that researchers, in planning their areas of study, will keep the eventual consumers of the fruits of their labours predominantly in mind, not only in terms of the objectives but also in terms of the manner in which the results are presented, especially in the engineering journals. It is accepted that to advance the frontiers, a certain amount of academic work has to be undertaken but it would be reassuring to be able to recognize some eventual application of some of the esoteric treatises one sees in some journals. I sometimes have the impression that the aerodynamicists are still talking to each other too much and that the mystique is being unnecessarily perpetuated.

Perhaps today's symposium, the first of its kind to be directly associated with an international wind conference, will be the start of improved understanding and closer collaboration between the aerodynamicist, engineer and architect. Within the context of the complex nature of wind and the uncertainties inherent in materials, methods of structural analysis, fluid mechanics theories and in wind tunnel techniques, one should not lose sight of the objectives – to design the structure and its component parts to reasonable margins of adequacy consistent with economy.

Much has happened in the last 100 years since the Tay Bridge disaster but, even now, we still have in our minds similar thoughts to those of the Great McGonagall who, of that momentous event, wrote the 'immortal' lines:

> 'The stronger we our houses do build,
> The less chance we have of being killed.'

What price progress?

Barbican Arts Centre

Tony Stevens

The design and construction of the substructure

The Barbican Redevelopment Project for the Corporation of the City of London has been under consideration and construction for the last 25 years. Most of the scheme is now complete, including residential blocks for about 2000 flats, a hostel, a school and other buildings with the conspicuous exception of the centre-piece of the whole undertaking – the Arts Centre. The Centre itself, when complete, will comprise a 2000-seat concert hall, a 1250-seat theatre, a cinema, the Guildhall School of Music and Drama, 96 flats in a horseshoe-shaped block, restaurants, shops, library, art gallery, car park spaces and so on. This article describes the substructure for the Arts Centre proper, that is the concert hall, the theatre and the cinema, and because it is not directly relevant, and in the name of brevity, I propose to omit the detailed history of the development of the design.

However, to appreciate the problem there are a number of factors which must be understood. Firstly, whilst deliberations on the Arts Centre were progressing in the middle 1960's, the design and construction of the remainder of the project was proceeding slowly but inexorably towards completion. Consequently, when the time came to begin the design in 1968, the site assigned to the Arts Centre was surrounded by buildings which were either in course of construction or completed. Secondly, as the brief for the Arts Centre developed and the client defined his requirements, the volume of the building increased several times over and above that anticipated when the surrounding development had been designed. Thirdly, the architectural design of the project did not admit of any of the considerable bulk of the Arts Centre above podium level – the podium is a level of general pedestrian circulation over most of the Barbican Scheme, about two floors above the street. What could not go up had to go down. Thus the stage was set for the problems involved in designing and constructing an extremely large excavation in London clay in the centre of a recently-completed development which included two 42-storey tower blocks and a re-aligned underground railway.

The architects, Chamberlin, Powell & Bon, prepared a report in April of 1968 for the client. The proposals for the new Arts Centre were accepted and we were authorized to proceed with the preparation of the design. At this stage the volume requirements for the building exceeded even the final version. There was a conflict between the need to simplify the substructure profile to facilitate design and construction, and to minimize the space requirements. In the end, the minimum space requirements prevailed, chiefly on grounds of apparent cost. The substructure profile now reflects the minimum requirements of the auditoria for the concert hall, the theatre and cinema with their associated plantrooms and the foundations for the 96 flats in the block above in an arrangement of substructure illustrated in Figs. 1 and 2. As a consequence of these requirements we had to make a design which would allow the removal of 250,000 yds.^{3} of mostly clay soil in a hole up to 70 ft. below street level, and in plan of dimensions roughly 500 ft. by 200 ft. without adversely affecting the foundations of the surrounding buildings, and, most importantly, without disturbing the piled foundations for the two 450-ft.-high tower blocks.

A soils investigation had been carried out in 1961 in anticipation of starting work on the whole scheme. A supplementary and more detailed investigation was made in 1970. The results showed the now familiar sequence of soils, namely (from the top downwards): made ground up to 5 ft. thick; compact gravel – about 10 to 20 ft. thick; London clay – about 70 ft. thick; Woolwich and Reading Beds – about 35 ft. thick; Thanet Sands – about 70 ft. thick; and then the chalk. Across the site we had found the water table occurred usually about 2 to 3 ft. above the top of London clay, but, because of pumping from the chalk over a number of years in the past, the water pressure profile through the London clay was about 50 to 75% of hydrostatic pressure, and disappeared entirely at the bottom of the clay bed. We took advantage of these conditions in the design.

An important factor in our considerations was the over-consolidation of the London clay, the result of an overburden of some 1000 ft. of soils now long since removed.

Note
As this job was designed in imperial units, we have left all dimensions and quantities expressed in this way.

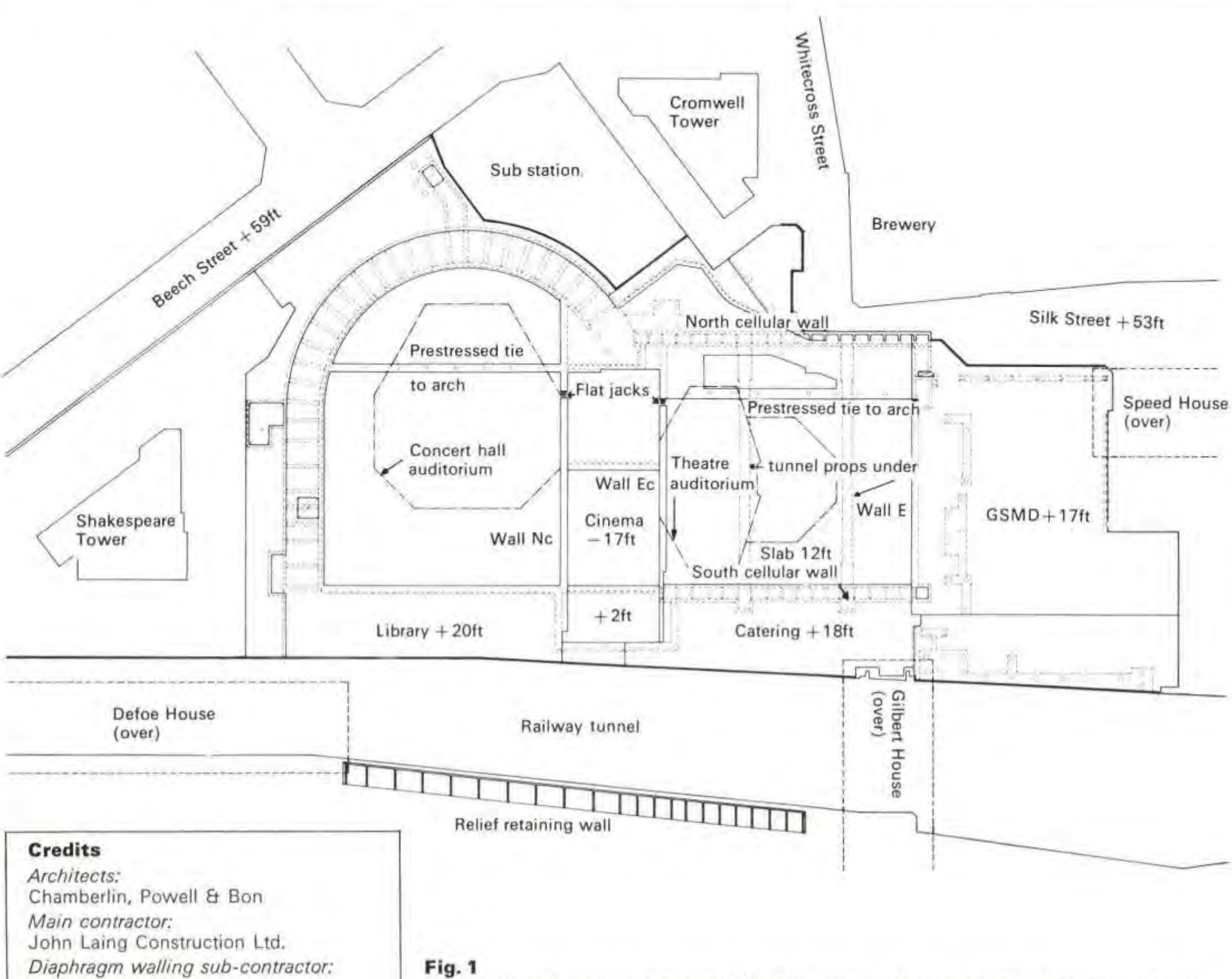


Fig. 1
General plan of accommodation at 20ft. level to show the concert hall, theatre and cinema

Credits
Architects:
Chamberlin, Powell & Bon
Main contractor:
John Laing Construction Ltd.
Diaphragm walling sub-contractor:
Tarmac Soletanche

The residual effect is that the coefficient of earth pressure at rest, K_o' (the ratio of horizontal effective soil pressure to vertical effective soil pressure) is greater than unity. It is in fact about 1.5 to 2.0. Another important factor is the permeability of London clay and the influence that this has on the speed of dissipation of pore water pressure set up by strains to which the soil may be subject. Given drained conditions in the soil, where the pore water pressure and ground water pressure are the same, the clay can be approximately represented, for the purposes of computing horizontal ground pressures, by a frictional material having a ø value of about 25°. Alternatively, if the negative or suction pore water pressures set up by shearing deformations at constant volume could be depended upon to hold, then the clay could be assumed to possess the considerable shear strength usually associated with London clay and characterized by measurements taken in the undrained triaxial shear test. The trouble was, and for that matter still is, that the period over which it may be safe to assume that the undrained conditions will apply is extremely uncertain. Such conditions may not be considered a sound basis for design for a project where labour disputes and delays are not unknown. Moreover, the question is not trivial, since the difference in the soil pressures developed against retaining walls in these two extremes of condition is very large with significant effect on the design.

Discussions which took place in the early days of the development of our design between Geotechnics (David Henkel, Brian Corbett, *et al*) and us in Structures 1 were influenced by the speed with which slips in clay soil had occurred at Bradwell Power Station and at other locations with which we in Arups were familiar. It was considered possible, although not very likely, that the higher soil pressures associated with drained conditions within the London clay could be mobilized in less than three weeks from the time of occurrence of the original soil strain, when the excavation was cut. Taking all the conditions of the project into account, we decided that the equilibrium calculations that we were going to make, both around retaining walls and in slip circle analyses, should be based on the drained as opposed to the undrained properties of London clay.

There have been a number of stoppages in the construction of the substructure and on occasion we have been more than glad that the stability of certain sections of the project were not, in theory at any rate, dependent upon time. But in recognition of the undoubtedly conservative assumption, we argued that a stability factor of 1.5 (ultimate resisting forces/driving forces) against rotation and sliding in retaining walls and slip circles was adequate until the project was complete, and the full vertical loads applied, when we considered that it was necessary to show that the factor was never less than 2.0. Thus design studies took two parallel lines of investigation: the equilibrium of soils/retaining works; and the likely movements on the removal of the excavation, taking account of the restraint offered by the substructure.

Predictions of movement

Our first predictions of the movement which would occur on excavation were based on the assumption that the horizontal pressures in the London clay would be reduced from the high level associated with over-consolidation to that corresponding to active soil pressure behind the substructure retaining walls. We did the sum by guessing the horizontal distribution of stress normal to the walls, by estimating the modulus of elasticity for the clay from our previous experience and by assuming the change in stress described above. The maximum values of movement obtained were of the order of 2 in. Geotechnics followed this up with a programme of finite element analyses which were carried out on their behalf at the BRS and later at the University of Wales,

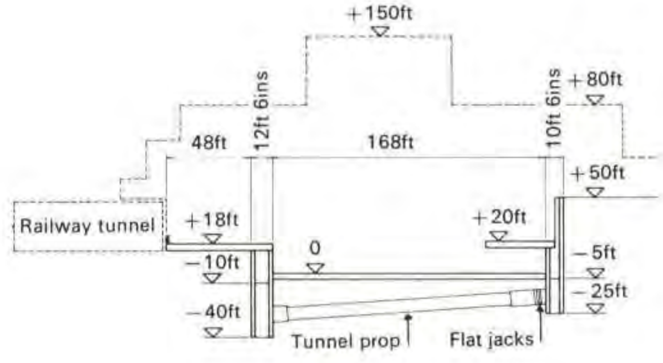
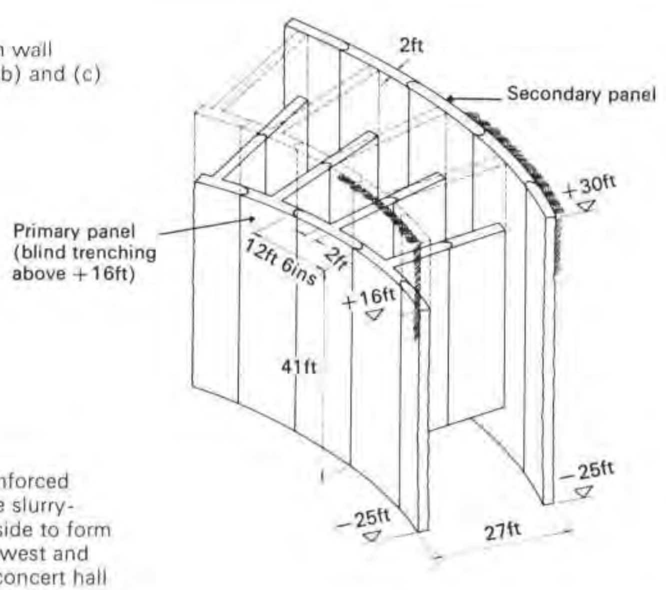
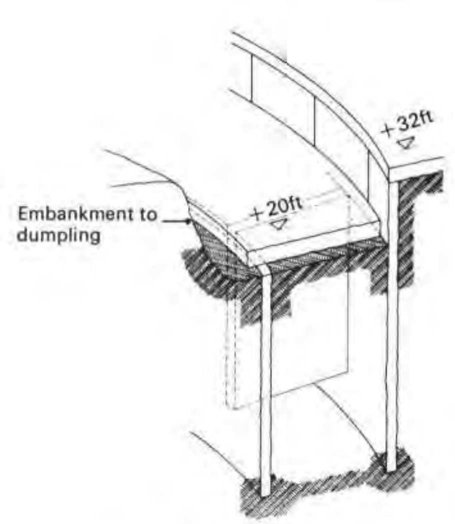


Fig. 2
North/south section through theatre showing north and south cellular walls and walings, 18ft./20ft. level slab walings, tunnel prop under ground slab and profile of structure above

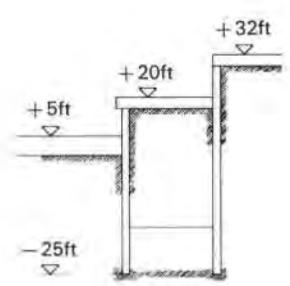
Fig. 3
Concert hall: west and north wall construction sequence (a), (b) and (c)



(a)
Stage 1
T-shaped walls of in situ reinforced concrete formed in bentonite slurry-filled trenches, cast side by side to form cellular retaining wall along west and north (curved) sides of the concert hall



(b)
Stage 2
20ft. level ground slab installed, retaining maximum amount of dumpling to restrict forward yielding of the wall and to maintain stability. Slab subsequently acts as waling spanning between north/south props (wall Wc and west side construction)



(c)
Final condition

Swansea, which provided more sophisticated answers. But even our unrefined predictions showed that the amount of movement could not be safely imposed upon the surrounding buildings. We could not reconcile ourselves to allowing the bodily horizontal movement of the two tower blocks which appeared to be implied. Moreover, the apparent flow of soil under the tower blocks, in the expansion which would result from the removal of soil from the Arts Centre, might destroy the carrying capacity of the piles. We could not allow conditions to develop where there was a risk that this prediction might come true.

A positive system of restraint

The Firm's previous experience of big excavations in similar circumstances indicated the benefits which would accrue from providing a positive system of restraint as opposed to relying solely on the behaviour of clay soil which was notoriously difficult to predict. To be of any use, such a system had to be capable of application at short notice, capable of supplying a very considerable force and would inevitably be substantial and costly. To avoid uneconomical duplication, the use of as much as possible of the permanent works for such purposes would be indicated.

So it was decided that the range of horizontal stress change in the soil around the Arts Centre excavation would have to be limited both during and after construction by the physical restriction of horizontal movement afforded by the substructure. After considerable deliberation we determined to attempt to hold the stresses behind the retaining walls to a level where the K_o' (effective stresses) value was approximately equal to unity. Why? Well, for one thing, Geotechnics advised that this was the horizontal pressure which the greatest degree of practical restraint would be likely to achieve. They based this conclusion on measurements taken behind well-restrained earth faces, of which the best example was underground tunnel linings. Another reason for our decision was that the forces involved at this level of restraint would be very large indeed. It did not seem either reasonable or feasible to attempt higher levels of restraint. Even then some movement, to allow relief of pressure in the clay, would have to take place. It was estimated to be a horizontal forward movement of about .03 ft., which we thought would be acceptable. It seemed, therefore, that the best we could do was just good enough.

Accordingly we adopted two techniques, namely: to pre-load the substructure works against the soil before allowing unrestricted excavation in front of the face; or to unleash the pressures behind the face by such small geometric increments followed by immediate stabilization that the cumulative effect of the soil movement would be small.

Pre-loading the substructure works

In the more usual substructure design it would have been possible to provide restraints at floor levels, before the soil was removed in stages. But this procedure would not necessarily have resulted in the limitation of movement that we needed, and in any event, the two major auditoria had no floors or other structure which could be used in this way. We examined the requirements for a structure which could provide the restraint, taking into account the forces which would have to be resisted if equilibrium were to be maintained under $K_o'=1$ pressures. The deepest excavation closest to a tower block (Cromwell Tower) occurred in the north of the theatre. Another critical face adjacent to Tower Block 2 (Shakespeare Tower) was located at the west and south of the concert hall. The least critical face was the east face of the theatre adjacent to the Guildhall School. Finally we decided to prestress the north/south faces apart with as much effective structure as could be introduced within the planning requirements for the three auditoria. The control of movement in the

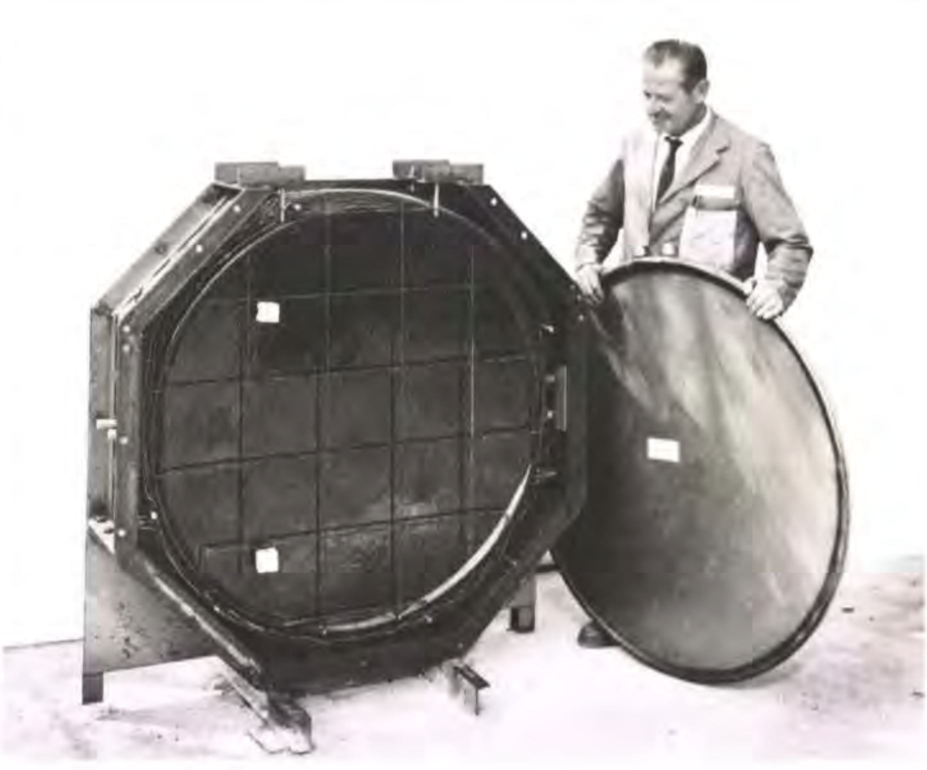
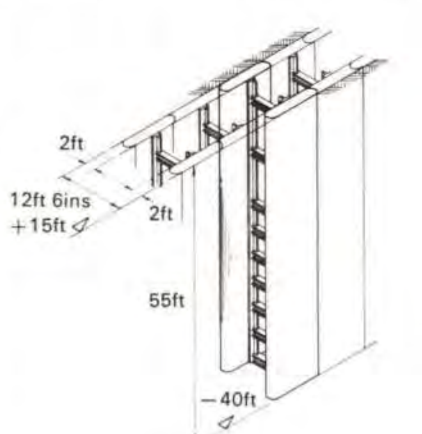


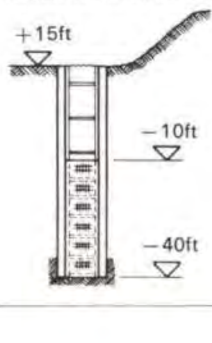
Fig. 4
A typical jack (Photo: PSC Ltd)

Fig. 5
South diaphragm wall of the theatre: construction sequence (a), (b), (c) and (d)

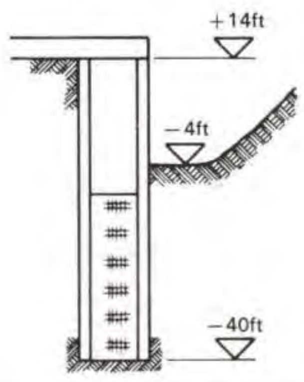
(a)
Stage 1
Walls formed in bentonite slurry-filled trenches; web ladder units of structural steel installed in primary trenches; flanges of in situ reinforced concrete, cast in secondary trenches, side by side to form cellular retaining wall 180ft. long.



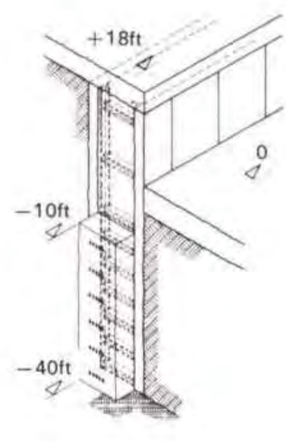
(b)
Stage 2
Excavation of 'cells'; tendons threaded through ladders for prestressed waling spanning between walls E, Ec and tunnel props



(c)
Stage 3
Ground slab cast at +18ft. to form waling spanning between walls E and Ec. Theatre ground slab cast



(d)
Final condition



east/west direction adjacent to Tower Block 2 would rely on the construction of the west face of the concert hall retaining walls under a controlled sequence.

At first our idea was to introduce jacks to pre-load walings, struts, etc., wherever a significant movement seemed likely to occur, but it was soon clear that such additional complexity in a layout which was already complicated might jeopardize feasibility of construction. We therefore reduced the points at which forces would be applied to the substructure to an absolute minimum number – five. The final form of the design is as follows.

Final form

Restrictions to movement at the north of the concert hall are provided by a 200-ft.-span, circular arch which could be formed readily within the foundations for the horseshoe flat block at 20 ft. OD level. Since the lowest level of excavation in this area is higher than elsewhere, one level of restraint is sufficient. The restraining forces spread into the soil through a system of two rings of diaphragm walls, 2 ft. thick (see Fig. 3), which in this construction are propped apart by radial diaphragm walls in order to further limit local deformation. The radial struts retained the two rings during the excavation for the construction of the 20 ft. OD level slab, which took place when the general level of the soil was at 35 ft. OD.

The reactions from the arch passed down the west side of the concert hall through a similar cellular diaphragm wall arrangement (Fig. 1) whose bulk persuaded us to omit provision for pre-loading. On the east side of the concert hall the reaction from the arch passed southwards along a strut formed by a series of 5 ft.-thick diaphragm wall panels in line. The thickness of the wall was determined by the dimensions of the jacks which it was practical to employ. At the south face of the concert hall the reaction forces from the arch are balanced against a piled wall to which forces are distributed through 175 ft.-span waling beams built within the foundation slabs at −2 ft. OD and 20 ft. OD levels.

At the west face of the concert hall excavation the aim was to excavate in narrow strips, about 20 ft. wide. Whilst the excavation was open, restraint was offered by the 20 ft. OD level foundation slab spanning horizontally between completed retaining wall at the south and the unexcavated soil to the north. After excavation further forward movement was prevented by the completion of the concert hall foundation slab.

Theatre auditorium

In the theatre auditorium the north face of the excavation was much deeper than elsewhere – about 60 ft. – and the forces much higher. Two levels of restraint were necessary. We were able to form a waling in the form of a horizontal tied arch of 175 ft. span in the reinforced concrete slab located at 20ft. OD level (Fig. 1). Its reactions were supplied by two struts, against 5 ft.-thick diaphragm wall panels running north/south. The second restraint was at low level, −15 ft. OD. We examined various alternatives and finally chose two reinforced concrete props formed in tunnel. The walls to the north and south of the theatre (Fig. 5) needed to possess both vertical, and at low level, horizontal bending strength. We met this requirement in our original design by specifying a series of I-section diaphragm wall panels side by side which would provide the vertical bending strength. The wall thicknesses were to be 2ft. The panels extended down to −25 ft. OD level. We proposed to excavate within the walls between the panels, introduce horizontal prestressing cable, fill the cells with concrete, then stress the whole together horizontally below 5 ft. OD level.

In order to limit the deflections in the 20 ft. OD level arch at the north of the theatre on loading, we decided to prestress the tie in a sequence which encompassed the raising of the tie force and the forces in the jacks at the reaction walls, at one and the same time.

At the south face of the theatre we designed another I-section cellular diaphragm wall, whose overall thickness and depth had been arranged to mobilize the passive resistance which is necessary to balance the soil forces from the north wall, applied through the north/south struts – that is the diaphragm walls and tunnels.

In the cinema area, between the two auditoria, where the excavation for the cinema auditorium itself went down to −22 ft. OD, a design of similar character was developed by which restraint is provided with horizontal beams within foundation slabs at +18 ft. OD level, at about 0 ft. OD level and finally at −17 ft. OD level. The side walls to the cinema excavation were formed with a combination of 4 ft. diameter bored piles and the 5 ft. thick diaphragm walls which doubled as struts in the north/south direction (see plan, Fig. 1).

These arrangements provided for at least one level of horizontal restraint to each cross-section of basement wall during construction, and more when the substructure was complete. The forces for which the systems were to be designed were determined from the requirements for rotational equilibrium about the fixed restraint points. As described above, soil pressures were intended to be those associated with the $K_o'=1$ effective stress values on the active side, taken with a passive pressure consistent with drained soil conditions. Equilibrium satisfied under these assumptions would not be critical to the passage of time and would meet the requirements for limitation of movement. Systems of resistance against these pressures could be mobilized in the completed structure, but it was not everywhere possible to achieve adequate factors of safety at intermediate stages of construction when equilibrium had to rely only on the contribution from the passive resistance of the clay soil.

Fall-back arguments

We had, therefore, to consider what fall-back arguments there might be to justify the design at all stages of the work. One possibility was to question the need for a factor of safety much greater than 1, when we had assumed the lowest value of passive resistance consistent with slow, long-term strain to failure where factors for stability could be improved by a slight forward movement which would reduce the driving and increase the resisting soil pressures, and where the maximum passive pressure would be mobilized in the overconsolidated clay soil by the removal of spoil from the excavation, with little associated forward movement at the retaining wall face. Formally there was no problem with the factor of safety since the codes do not require a driving pressure in excess of active to be considered, but from our point of view, these arguments left little margin for error in our prediction of soil forces.

If we needed a margin it might be argued that under compression, from forces on the active side, fissures in the passive zone would tend to close. The permeability of the soil would thus be reduced, and strain would occur under undrained conditions. Driving forces would be considerably less than those generated under an assumption of a drained soil. As before, pressures close to, or at, the passive limit would be developed to provide a higher factor of safety against rotation with little horizontal movement. The disadvantage was that the level of pore water pressure set up would be indeterminately dependent on the passage of time under conditions where the speed of construction could not be reliably controlled.

Another possibility was to accept the soil strains associated with a change in soil pressure from that which obtained before excavation to the maximum active under fully drained soil conditions. The decrease in soil forces would reduce the problems of equilibrium but, of course, with a risk of unacceptable movement. It would be necessary, therefore, to limit the volume of soil affected by such strains so that the absolute value and extent of horizontal movement would be relatively small.

In fact we had to employ all of these arguments to complete the design. In general only the south-west wall of the concert hall area was sequenced to limit soil strains and to protect foundations for the Shakespeare Tower. Throughout we enjoyed either an adequate factor of safety against our predictions of soil behaviour at failure or the ability to apply a reserve of positive restraint forces to the soil through the substructure.

Local calculations of this type employing linear soil pressure diagrams in simple systems of equilibrium provided the means for estimating the necessary restraining forces and the bending moments and shears against which the elements of the retaining walls could be designed. The general level of equilibrium of soil under and around the substructure was confirmed by a slip circle analysis carried out on a number of typical cross sections, using Bishop's Simplified Method. Once again the drained properties of the soil were employed in both the overlying gravel and the London clay. The weight of the surrounding buildings was included where appropriate, and so was the influence of the application of horizontal forces on the friction in the slip circle. The three-dimensional effect of the curved faces of the excavation were not taken into account in the sections studied.

As an example, the minimum factor against rotation, with no applied restraints, in a typical cross-section through the north-east wall of the theatre is less than 1 – that being the ratio between moments created by the driving forces and the moments generated by the resisting forces. With forces applied at 20 ft. OD level and −15 ft. OD level through the substructure, the factor against rotation was raised to about 1.7, and, of course, the addition of the dead weight of the Arts Centre after construction raised the factor yet higher.

Movement after soil removal

The movement likely to occur upon the removal of soil in the excavation, in typical retaining wall sections, assuming plane strain conditions with and without the restraining forces afforded by the substructure, was estimated by elastic analysis using the finite element method. To make this feasible it was necessary to know or to assume the conditions of strain and of stress, the elastic properties and Poisson's ratio of the soil and the history of stress conditions in the soil before excavation. Of course, doubt attaches to each of these necessary assumptions which might call the whole procedure in question, but no better method exists by which an estimate of movements could be made and which was reasonably compatible with all the factors we wanted to take into account. On that basis the programme proceeded.

Sections were assumed to deform as if the material were linearly elastic under plane strain conditions, on the grounds that such analyses had proved reasonably effective on previous occasions. The elastic properties of the soil were taken from values which were deduced from the movement and analysis of the Britannic House excavation (in Moor Lane, close by), assuming that the soil would have a Poisson's ratio approaching 0.5 for immediate movement. The clay soil was assumed to have a modulus of elasticity which varied through the bed from 0–1250 tons/ft.^{2}. The value assumed in the Woolwich and Reading Beds was very high – 3000 tons/ft.^{2}. The sections for analysis were made to extend far enough beyond the face in which we were interested to render the boundary conditions non-critical. The forces applied to the surfaces of the soil sections were those which represented the relief of vertical and horizontal soil pressure plus those expected to be applied through the substructure.

The final analysis was carried out under the supervision of Geotechnics at the University of Wales, Swansea, in 1971. Without restraint forces, the maximum horizontal movement of the face was predicted to be of the order of 2 in. with restraint, the movements were estimated to be of the order of 0.03 ft. Earlier, in 1970, David Henkel produced a note on the immediate horizontal movements of London clay in excavations which gave similar results and on which we relied to supplement the finite element program.

You will have noticed an inconsistency in these arguments. The equilibrium calculations assume that the undrained condition cannot be relied upon, whereas the deformation calculations assume they apply. It was a question of expediency and it was difficult enough to do the sums assuming the soil to be linearly elastic. It was not possible to perform the exercise of modelling the non-linear behaviour of the soil with time. So the possibilities were either that the movements might be greater for the same restraint, given sufficient time, or that the forces required for restraint might be somewhat higher than we had assumed. But we believed that the level of forces we had taken would be the greatest likely to develop under effective horizontal restraints. Secondly, once completed, the substructure was not critical within a reasonable range to the forces applied to it. This then was the basis of our design.

Movement of the north and south walls

To pre-load the substructure system horizontally in the north/south direction, we proposed to introduce groups of 1.08 m diameter flat jacks to be supplied by PSC Ltd. in all three of the 5 ft.-thick diaphragm walls, and at the north and of both the tunnel props. We knew, or could guess, the likely elastic shortening in the substructure. What we did not know precisely was the amount by which the north and south walls would want to move together or apart, given the forces applied. The actual movement would depend upon the previous history of stress changes, on the pressures and their distribution across the earth faces, and on the deformation of the structural system. In the theatre, for example, we had at least to consider the possibility that the north wall would finally move north rather than south. The elastic shortening of the 5 ft.-thick diaphragm walls under the axial forces in them would be about 0.02 ft. more including creep, and the elastic shortening in the tunnels would be about 0.08 ft. The deformation in the walls on the south side of the theatre under the action of the passive pressures and the prop loads might be as much as 0.02 ft. So the opening of the jacks could be about 0.07 to 0.1 ft.

Each of the individual flat jacks was capable of a maximum opening of about 0.08 ft. We decided to concrete them in stacks of four so that there would be a reserve on the maximum extension, even supposing that one in the stack failed. We specified a monitoring system which would automatically maintain the level of load required over the considerable period of time that we intended to keep the jacks active. A warning would be given if the pressure in the jacks tended to vary beyond set limits. PSC had never made circular jacks of this size before and instituted a programme of testing to satisfy themselves that their design would perform satisfactorily. We were concerned that a relative movement tending to shear the jack stacks might occur because of, for example, the shortening due to prestressing in the 20 ft. OD level arch or because of lateral soil movement at the tunnel props. Accordingly, each stack included a sliding plate, PTFE-coated, which would allow movement to occur under a shear load without affecting the capacity of the jacks (Fig. 4).

The sequence of construction was as follows. It was intended that all of the north/south propping/waling system (see Fig. 1) should be constructed and pre-loaded before the majority of the soil in the auditorium below 35 ft. OD was removed. Once the forces in the props had been raised, the way would be clear to take out the soil in the theatre auditorium completely and to begin construction of the foundation slab at level 0 ft. OD. In the concert hall the construction of the foundation slab against the west wall adjacent to Tower Block 2 was to proceed northwards, as described above, in increments of excavation about 40 ft. across, further divided into 20 ft. increments at the west face. The excavation of the cinema could commence after the ring of foundation slabs surrounding it at approximately 0 ft. OD level had been completed and the lateral restraint of the side walls could be assured. After that the excavation and construction of the −17 ft. OD level cinema foundation slab could be carried out.

In 1970, when John Laing had been appointed in the first stage of a two-stage tender for a target type contract, negotiations were opened with Tarmac-Soletanche who had been the selected sub-contractors for the diaphragm walling. Tarmac-Soletanche agreed that the proposals for the cell units both in the concert hall and the theatre were feasible. The T-shaped panels in the concert hall shown on Fig. 3 were modifications that they suggested. But although they said that they could form the I-units in the two cellular walls in the theatre, they preferred to make each web as a precast unit installed in cementitious mud and to form the flanges as separate panels – that is if our design could be suitably modified.

The excavation between the diaphragm wall skins to form the north and south theatre retaining walls constituted a face of approximately 200 ft. × 70 ft. on the north face of the theatre across which movements would be considerable unless precautionary measures were taken. An important purpose of the webs in the cellular walls, therefore, was to hold the north and south faces of the cells apart in order to restrict movement. Finally we agreed to substitute a series of steel ladders (10 ft. 6 in. centre to centre) for the concrete webs to act as prop frames in a way familiar in coffer dams (Fig. 6). The ladders were to be installed in cementitious mud after which the 2 ft.-thick outer diaphragm walls were to be formed against their flanges. This arrangement also served to reduce the problem of forming or cutting holes through the concrete webs to pass the horizontal prestressing cables. We could not adopt this design at the east end of the north cellular wall in the theatre, where the diaphragm wall had to cantilever above the 20 ft. OD level slab to restrain the street at level 53 ft. OD. Nine T-shaped panels, spaced at 10 ft. 6 in. centre to centre, therefore remained in reinforced concrete diaphragm walling. It was considered impracticable to form holes to pass the cables, and the openings were subsequently cut after excavation had been completed.

Construction joints in most of the 2 ft.-thick diaphragm walls were formed as usual with tubes. The north/south prop walls at 5 ft. were too thick for the adoption of this method and Tarmac-Soletanche proposed that the joints should be made with precast panels sunk in cementitious mud. The concrete units would be pre-coated with a bond-breaking wax to facilitate the removal of the mud in preparation for casting the diaphragm wall panel concrete against them. At the time Tarmac-Soletanche said that this method was reliable, and that they had experience of the use of it in their *Panosol* system. Later these joints were to give serious trouble.

The work on the main contract started on site in April 1971 on the south side of the LTE railway between Barbican and Moorgate Stations. The construction was a retaining wall whose purpose was to relieve the tunnel of the unbalanced soil loads arising from our intention to excavate to the level of the tunnel foundations on the north side only. An important condition of service was that there should be no contact between the railway and the Arts Centre structures so that there should be no possibility of vibration, and therefore noise, passing between the two. This relief retaining wall provided the best solution.

This work and site clearance continued until early January 1972, when the diaphragm wall contract began. We had recognized that the complicated panels would require about four or five days to excavate and concrete, but we naturally wanted the panel construction time kept to a minimum. Tarmac-Soletanche for

Fig. 6
The completed excavation of the south cellular wall in the theatre, showing the steel ladders (Photo: John Maltby)

Fig. 7
The interior of the prop tunnels (Photo: John Maltby)

their part wanted to work long hours during the day to meet these requirements and their programme. The site is surrounded by the occupied flats of the remainder of the scheme and some residents objected, vociferously, to the noise that Soletanche made, particularly late in the evening and early in the morning. A noise-abatement notice was served but, luckily for us, Tarmac-Soletanche were never actually stopped.

After work had been under way for a few weeks of the 36-week programme, it became clear that the character of work on the shaped panels was noticeably different in a number of ways from that with straight diaphragm wall panels. Almost all the differences related to the length of time each individual panel excavation remained open, which in turn was related to the panel shapes. Nonetheless, construction of most of the panels was surprisingly trouble-free, although there were some exceptionally difficult ones. Technically the work went well on the whole, with one exception described below, and Tarmac-Soletanche finished within their contract period, with the national building strike of 1972 intervening.

In November 1972, Tarmac-Soletanche left the site. Work on the 20 ft. OD level arches and walings had begun. All the 20 ft. OD level ring around the two auditoria had to be completed, including the difficult construction at the north of the theatre, and the capping to the 5 ft.-thick diaphragm wall struts, before stresses could be raised in the north/south system. The work took nearly two years to do, about twice as long as planned. Labour disputes were the chief components in this delay, accounting for about half. But work in the cellular walls took longer than was programmed, principally because the work was unusual and demanding.

By April 1973, the excavation for the cellular wall at the south of the theatre was complete (see Fig. 6). Excavation proceeded within the cell wall to the lowest level (−40 ft. OD) and the two tunnels lined with precast concrete units were driven from adit cells at two points along the wall (Fig. 7). The lower halves of the tunnels were concreted in three lengths by normal methods. The upper halves were grouted in one operation with cement mortar of strength 6000 lb./in.^{2} injected by Colcrete Ltd. Both tunnels were filled by the end of July 1973.

Difficulties

The work on the cellular walls was slow, chiefly because of the preparation of the inner surfaces. We had designed the prestressed infill concrete to be composite with the two outer skins because we needed both the strength and stiffness and, in the case of the wall at the north end, we were confined to a thickness of 10 ft. 6 in. over the outer faces of the diaphragm walls. The interface stresses were such that we needed nominal reinforcement for which we provided bars in bands at 2 ft. centres to be pulled out of the faces of the completed diaphragm walls after the excavation in the cells. In the event, some of the bars were buried more deeply than had been intended because of overbreak and mispositioning of cages.

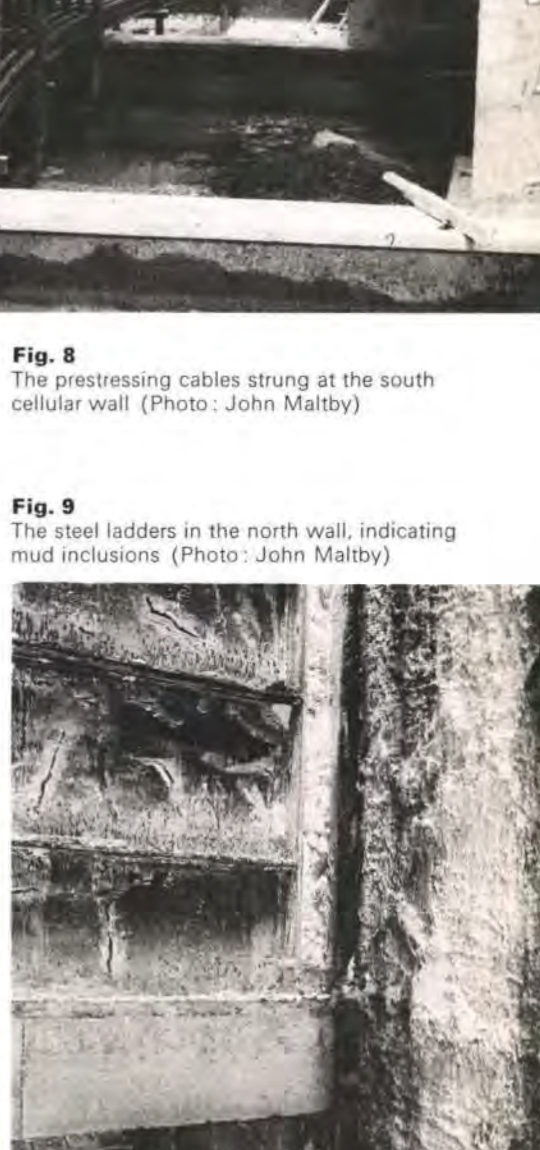
The reinforcement for the composite cell walls spanning across the tunnel props to the 5 ft.-thick diaphragm walls at the end was provided by prestressed cables. After some discussion with Laings, we chose the seven-strand 0.7 in. diameter *Dyform* plastic-covered unbonded cable made by Bridon, to replace the sheathed and grouted system we had planned originally. We believed that the loss of strength in the wall, which we could afford, would be more than compensated by the reduced risk in concreting and grouting the ducts after stressing. Eventually the first lift of cables was strung (Fig. 8) and concreted, and afterwards the work went relatively smoothly to the completion of construction in the southern cellular wall of the theatre by February 1974.

Fig. 8
The prestressing cables strung at the south cellular wall (Photo: John Maltby)

In the southern cell wall to the theatre there had been some cementitious mud inclusions between the steel face of the ladder props and the concrete of the outer diaphragm wall skins. The scraper that Tarmac-Soletanche had employed had wandered away from the face. In the south wall this factor had not been significant and there had been no trouble. When the north cellular wall was excavated, however, the situation was much worse. Several of the ladders had 4 in. and 5 in. inclusions of mud between the steel and the concrete (Fig. 9). We could not be sure that the pressure of the soil against the outside of the diaphragm wall, whatever it was, could be safely resisted. Consequently we had to give instructions for the casing of some of the steel struts in concrete, in order to form a secure prop and to ensure the stability of the excavation. There was another difference between the south and north walls. The south wall had been relatively dry, but water seeped in through the face and at the bottom of the north cellular wall, fast enough to require constant pumping. Moreover, the irregularities in the face were greater in the north wall than they had been in the south, and this was an important reason why construction could not proceed at the speed expected. Nonetheless the wall was stabilized. The inner surfaces were prepared using in some cases *Duplex* anchors to replace pull-out bars which were embedded too deep in the concrete. All four lifts of the waling were concreted by the middle of April 1974 but unfortunately this was not the end of it. Cement grout had seeped into some of the cable anchorages and the tendons could not be spread and stressed until it had been removed. Laboriously the grout was chipped away and finally the strands were pulled up to the loads and extensions we had expected, and both cell walls were complete.

In the meantime, another problem had been uncovered. Evidence had been appearing here and there that the construction joints in the 5 ft.-thick diaphragm walls were not as they should be. When two joints were exposed in the east wall of the concert hall, it was clear that, because the scraping techniques of Tarmac-Soletanche had not been fully effective, at least some of the construction joints would have inclusions of cementitious mud. After some deliberation we concluded that we could not rely on these inclusions to transfer the axial loads in the struts, and we would have to institute a programme of remedial work to clear and regrout the joints. How this was done is a story in itself. Suffice to say that all the joints in the 5 ft.-thick walls were inspected and those which were defective were repaired.

Fig. 9
The steel ladders in the north wall, indicating mud inclusions (Photo: John Maltby)



While this was going on, the flat jacks had been installed both in the walls and at the north end of the two tunnels in access shafts prepared for the purpose. The prototype jack tests had been completed successfully and the work of installation was without event.

We had made provisions for measuring the movements which would occur when the system was pre-loaded, and when the soil was removed from the excavation. There were lines taken between stations outside the influence of excavation from which offsets for datum points could be measured. Plumbing tubes were buried in the cellular diaphragm walls at the north and at the south of the theatre, and there was a point at which soil heave could be

Fig. 10
The site as it is today (Photo: John Maltby)

Fig. 11
Architects' impression of the completed Arts Centre (Photo: John Maltby)

measured in the middle of the theatre auditorium.

You may imagine that there was a certain amount of apprehension as the date for jacking – late September 1974 – approached. To begin with the system depended upon loads and movements which derived from the somewhat uncertain estimate of soil pressures at the north and south of the system. The structures involved were large and not amenable to fine calculations of their behaviour. We had argued for example that the 20 ft. OD level arch was flexible in relation to the soil behind the north wall of the theatre, but there was one stiff point in the span which had to go northwards into the soil about 0.02 ft. if the loads on which the prestressing was based were to be credibly realized. We were saying by way of our design that we could predict the flow of force around an excavation about 500 ft. × 200ft. to a degree which would allow the reliable limitation of movement on two faces. To that end, in the theatre area alone, we were to apply a force of 11,000 kips to stress the 175 ft.-long, 20 ft. OD level arch tie and to provide 40,000 kips of preload on the jacks in the north/south props. I found plenty of scope for imagination of the consequences if much of this force wandered from the path that we had provided for it.

Jacking took place on 27 and 28 September 1974. The pulling of the cables and the raising of the pressure in the jacks was trouble-free and, more importantly, the movements that occurred were substantially what we had expected, not to say hoped for. The whole episode was a bit of an anti-climax for which I suppose we should be grateful.

The jack stacks in the wall expanded approximately 0.03 ft. Those in the two tunnels expanded about 0.05 ft. The plumbing device in the cellular wall indicated that it was remaining substantially upright under the influence of all this force. The 20 ft. OD level arch pushed the north wall of the theatre back into the soil about 0.02 ft. on average – more at the critical point in the span. The arch tie shortened fractionally and the sliding plates behaved satisfactorily.

That was all a year ago. Since then almost all the excavation has been completed and most of the foundation slabs constructed (Fig. 10). The extension of the theatre jacks continued until they were filled with resin in May and June of 1975. Whether the extension was due to creep in the concrete or in the soil was not determined. The wall jacks extended about 0.06 ft. The tunnel jacks extended to a maximum of 0.1 ft. There was a relative vertical movement between the tunnels and the wall under the influence of the 0.2ft. of heave in the centre of the theatre auditorium excavation when the soil was taken down to −4 ft. OD. The expected southwards movement of the theatre north wall did not occur. We can only speculate on the reasons. Maybe the level of accuracy of prediction of movement – a small amount in either direction – is as good as could be expected. Maybe the K_o' values in the clay were lower than we expected. Maybe the horizontal pressure in the soil was relieved by the installation of the diaphragm wall or the propped excavation to 20 ft. OD level which followed. All we know is that the result is satisfactory. We reduced the load in the tunnel jacks in two stages of 100 p.s.i. — about 5% each. The second reduction seemed to stabilize movement.

Geotechnics had been measuring the levels on Cromwell and Shakespeare Towers both before and after jacking and during excavation. There was no material difference either in the level or the inclination of the blocks. Measurements were taken to an accuracy of 0.001 in.

On the west side of the concert hall the excavation has proceeded slowly in the step-by-step construction intended to limit the movement adjacent to the other tower block – Shakespeare Tower. In fact the movements that have occurred at the face of the excavation have not been measurable.

In June of this year, the stress was raised in the jacks on the western most 5 ft.-thick diaphragm wall. Their extension was 0.025 ft., and they have continued to open as the cinema excavation has proceeded. The movement of the concert hall arch under the influence of this force, about 7000 kips, was northwards roughly 0.01 ft. but we expect the movement to be inwards radially when the last of the concert hall excavation at the north end is taken out.

Conclusion

The object of this section of the Arts Centre design, that is to limit movements due to the removal of soil, and to the construction of the foundations, has been achieved. It has been done within a system which is compatible with the minimum volume requirements and the detailed planning of the auditoria and other accommodation in the Arts Centre. The extent of the temporary works, not incorporated as a working component of the permanent works, is small. We have exposed ourselves to the criticism that the character of the work that we have had to design has not been conducive to speedy construction, but that may be a relatively small price to pay for the security of the tower block foundations.

First preheat your steel

Alan Denney

Fig. 1
A hydrogen-induced cold crack originating at the toe of the weld and located in the h.a.z.

Introduction

This paper is intended to explain in straightforward terms the purpose of preheat and the factors involved in setting preheat levels for the range of structural steels classified in *BS 4360*[1]. Preheat is applied to structural steelwork to avoid the phenomenon of *hydrogen-induced cold cracking.* The heat is applied to a strip of steel either side of the weld line prior to welding. By increasing the total heat input to the weld it slows down the cooling, avoiding undesirable metallurgical changes and avoiding the cracking phenomenon. The temperatures applied are modest, usually in the range 50° to 250°C, exact temperatures varying with the welding and material parameters.

The restriction in this paper to the common structural steels is a necessary one. Preheat is a process applicable to several different categories of steels, the reasons for its use differing depending on that category of steels under consideration. Limiting the range of steels under consideration permits generalized comment to be made for them which are not valid for all steels. For a comprehensive treatment of all factors for the majority of ferritic steels, Reference 3 is highly recommended.

Hydrogen-induced cold cracks

Hydrogen-induced cold cracks occur during the cooling of the weld, generally appearing within a few hours of the completion of welding but occasionally up to 48 hours later (Fig. 1). The critical temperature range is when the component is between 160°C and ambient. Not all structural steels in *BS 4360* are equally susceptible to hydrogen cracking. If all welding parameters are fixed, including thicknesses and joint configurations, there is increased tendency for hydrogen cracking with increasing strength of the steel. Thus grade 55 is more susceptible than grade 50, and grade 50 more susceptible than grade 43. Grade 43 is only susceptible to hydrogen cracking when large thicknesses are in use, material thickness being one of the parameters of importance. Grade 40, which is rarely used structurally, is the least susceptible of all.

Cracks, when they occur, are located in the heat affected zone of the parent metal (known as the h.a.z.). This is the area of the parent metal which has not been fused into the weld, but which has undergone metallurgical changes within the solid phase. This zone naturally occurs outside the fusion boundary and is readily detected in any etched microsection of a weld, as in Fig. 1. The cracks do not necessarily emerge at the surface of the steel and may be classified as toe cracks, root cracks or underbead cracks depending on location (Fig. 2). The cracking may be continuous or discontinuous and is difficult to detect other than by ultrasonic inspection techniques. Radiography is not a particularly good non-destructive testing technique for such cracks, nor are surface inspection techniques such as magnetic particle or dye penetrant methods.

Importance of hydrogen cracking to design engineer

If no supervision is exercised and cracks are allowed to develop, there is clearly a danger that these may propagate in service. The cracks could initiate brittle fracture, causing sudden catastrophic failure or they could provide sites at which fatigue cracks may propagate. In chemical plant such cracks can cause leakage or contamination. There is a consequent obligation on the designer to ensure that welding procedures eliminate the chance of hydrogen cracking. In part this obligation is met in, for example, the Arup standard structural steelwork specification which requires that all welding is carried out in accordance with *BS 5135: 1974*[2] and in specifying that low hydrogen electrodes should be used with the higher tensile steels.

Specification clauses apart, it is clear that emphasis should be on the prevention of hydrogen cracks. This is ensured by

(a) using the lowest strength grade of steel (e.g. grade 43 where possible)
(b) checking the proposed welding procedures by welding procedure test pieces
(c) ensuring that the welding procedures, once established, are worked to, and
(d) carrying out adequate non-destructive testing.

Repair welds will cost many times that of depositing a good weld first time and may obviously cause expensive delays on the overall construction programme.

The fabricator's view may be that preheat is expensive to apply but it is much more expensive if defects occur due to poor application. Even where preheat is to be used things can go wrong to prevent the attainment of the necessary levels. The equipment used may be inefficient or the wrong procedure may be applied. The welders may object to working under the hot conditions brought about by the use of preheat and not follow the intended procedure. Tack welds may be applied under the mistaken belief that being of temporary application they can have no permanent ill effects.

The cracking mechanism

Steel can exist in several different crystalline arrangements depending on the temperature. It undergoes a structural transformation in the solid state before melting, changing at relatively low temperatures from alpha steel to gamma steel. The solubility of hydrogen steadily increases with temperature undergoing some marked changes at the transformations from alpha to gamma and from gamma to liquid (Fig. 3).

Hydrogen can originate from two main sources. The first may be from water or moisture on the surface of the steel itself, especially if this is rusty. The second is from water chemically combined in the welding flux or shielding gas. In the welding arc this water is broken down to its constituents of hydrogen and oxygen, each of which is in its atomic (uncombined)

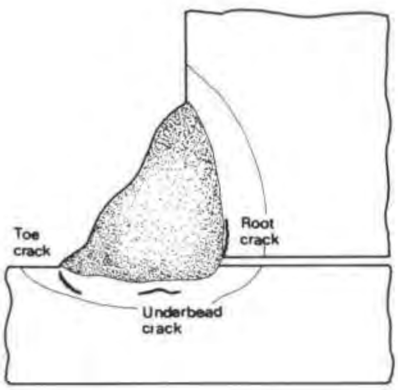


Fig. 2
Possible location of cracks

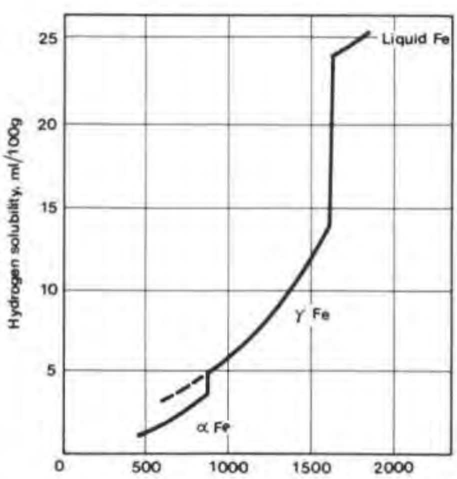


Fig. 3
Solubility of hydrogen in weld metal with temperature

state. Some of this hydrogen will transfer to the molten weld pool, in which atomic hydrogen is very soluble. As cooling occurs solubility falls and the weld becomes supersaturated in hydrogen. Much of it diffuses out but appreciable quantities are held in the weld and the heat-affected zone of the parent material.

The reason that hydrogen cracking occurs in the h.a.z. rather than the weld is that these two regions are generally metallurgically different. Typically it is quite normal to weld a low alloy steel of the grade 50 type with a mild steel electrode. Perhaps surprisingly this does not give any reduced yield strength in the weld metal compared with the parent metal. It does, however, result in different behaviour on cooling of the weld.

As Fig. 3 shows, there is a marked drop in hydrogen solubility from the gamma steel to alpha steel. In practice the weld metal transforms from gamma to alpha before the heat-affected zone. The large quantities of excess hydrogen migrate from weld metal to h.a.z. which is still gamma steel. On transformation of the h.a.z. (which occurs possibly only instants later) it becomes supersaturated in hydrogen. If cooling is relatively fast the hydrogen may not be able to diffuse out and, due to the metallurgy of the (low alloy) steel, a brittle metallurgical phase known as *twinned martensite* may be formed.

The combination of twinned martensite, supersaturation of the h.a.z. with hydrogen and a stress pattern (which may simply be the residual stresses resulting from welding) leads to a progressive cleavage of the h.a.z. microstructure. This is the hydrogen-induced cold cracking.

Factors in hydrogen cracking

There are thus four main factors which affect the response of a low alloy steel to hydrogen. These are:

(a) The potential hydrogen content of the welding process

(b) The rate of cooling of the weld (since this determines both whether twinned martensite will form in the h.a.z. and whether temperature and times are suitable for the hydrogen to diffuse out)

(c) The composition of the parent metal. This will determine whether a material can form twinned martensite under the cooling rate obtained in welding

(d) The restraint acting on a joint during the following welding.

In assessing necessary preheat levels, factors (a), (b) and (c) are particularly important; factor (d) is not considered. This is because it is not readily quantifiable at the present time. It is nevertheless of importance. Variations of restraint condition may well be a factor in explaining why one may *not* get cracking when it has been previously obtained on similar joints.

Hydrogen potential and the welding processes

The greater the hydrogen content of the weld metal the greater is the risk of hydrogen cracking, given that other factors are favourable for its appearance. Hence one of the methods employed by welding engineers to control it is to minimize likely weld hydrogen level by selection of particular welding processes and consumables (electrodes, gases or fusable fluxes depending on the welding process). Fig. 4 (after Coe[3]) illustrates typical weld hydrogen levels for different welding processes.

The welding processes of principal interest are manual metal arc, CO_2 and submerged arc welding. Manual metal arc welding is the name for the process using stick electrodes. These electrodes have flux coatings, several different coating types being in common use, each of different potential hydrogen content. Thus Fig. 4 shows different weld hydrogen levels for electrodes of class 6 compared with classes 2 and 3.

CO_2 welding is a gas-shielded, bare wire welding process, the arc being struck between wire and workpiece, the wire being fed continuously from a spool. The CO_2 provides the same role as the flux in manual metal arc welding of preventing oxidation of the weld pool.

Submerged arc welding is another bare wire process, this time using a blanket of granulated flux to provide shielding. It differs from CO_2 welding in being fully automatic and is used with larger wires. It finds application in larger structures and in shipbuilding. Flux-cored CO_2 welding is a variation on the CO_2 process in which the wire contains its own flux. It is not yet widely used.

The more susceptible a material, the less hydrogen can be tolerated. Fig. 4 hence explains why structural steelwork specifications frequently allow mild steel (which is not particularly susceptible to hydrogen cracking) to be welded using class 2 or class 3 manual metal arc electrodes but require that grade 50 steel, which is susceptible, should be welded with class 6 electrodes. Hydrogen-induced cold cracks cannot, however, be totally eliminated in all cases by choice of process and consumables alone and obviously other factors will decide which welding process may be suitable. Thus, preheat may not be eliminated by choice of process parameters, but clearly these will influence the preheat temperatures necessary to avoid cracking.

Heat input and rate of cooling

It is the rate of cooling of the heat-affected zone of the weld which determines whether it is hardened, and hence susceptible to hydrogen cracking or not. This, in turn, is dependent on the thicknesses and geometry of the materials being welded, *the thicker the material the more rapid the cooling and the more susceptible the h.a.z.*

The other major factor in rate of cooling is the total amount of heat supplied, both before (preheat) and during welding. In the assessment of necessary preheat levels this heat input from the welding process must be assessed. This is calculated per unit length of weld and expressed as arc energy. Thus, for a particular thickness of a particular low alloy steel, welded by a given process and using known consumables, control of cooling rate is achieved by varying the arc energy and preheat applied. Hence this controls the susceptibility of the steel to hydrogen cracking.

The composition of the parent material

Composition of parent steel is obviously a major factor in hydrogen cracking, and it has been indicated above that the structural steels may be loosely assessed as susceptible (grades 50 and 55) and less susceptible (grade 43). One may generalize further and say that grades 50 and 55 will require the use of low hydrogen processes and consumables (e.g. CO_2 welding or class 6 electrodes as in Fig. 3) and that grade 43 will not. Grade 50 is very likely to require preheating, grade 55 is more so and grade 43 less so.

The important difference between these steels is their susceptibility on rapid cooling to the formation of hard microstructures (particularly twinned martensite), in the heat-affected zone, which are susceptible to hydrogen cracking. This property of a steel is generally called its hardenability, the hardness produced being measurable by carrying out micro hardness tests.

The hardness achieved is thus a function of composition and cooling rate. Hardenability increases with carbon content of the steel and, since each alloying element makes a different contribution to hardenability, empirical formulae have been developed to express the total of their effect on a 'carbon equivalent' scale. That used in *BS 5135* for steels to *BS 4360* is:

Carbon equivalent =

$$C+\frac{Mn}{6}+\frac{Cr+Mo+V}{5}+\frac{Ni+Cu}{15}$$

All figures are expressed in percentages.

Stress level and restraint condition

All welds develop a residual stress pattern due to the thermal contraction of the weld metal and these will be intensified if the structure is a rigid one where the weld is under appreciable restraint. Coe[3] reports that hydrogen appears to lower the stress level at which cracking will occur, and the high degrees of strain which result produce higher risk of cracking for a given microstructural hardness. The stress act-

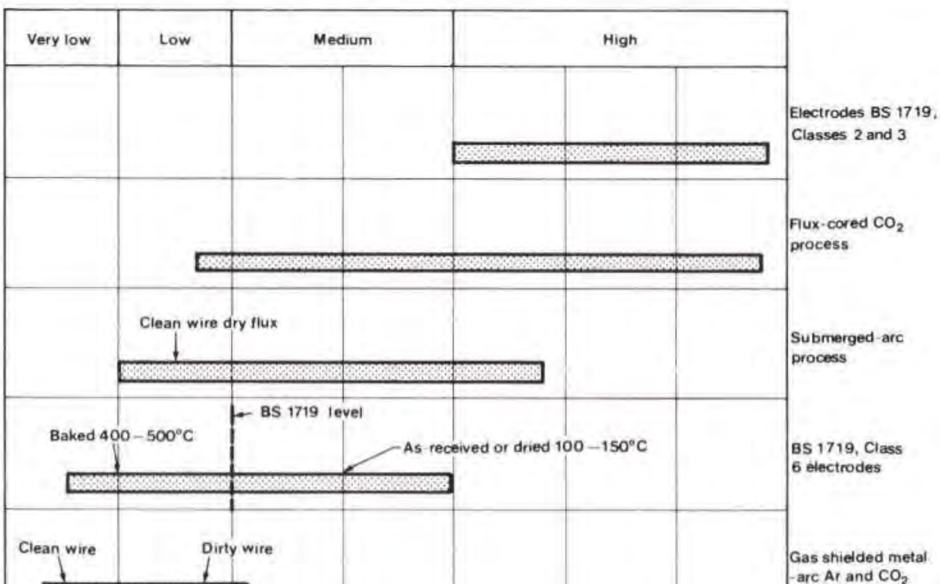


Fig. 4
Typical weld hydrogen levels

ing upon a weld is a function of weld size, joint geometry, fit-up, external restraint, and yield strengths of plate and weld metal. There is, at the moment, little data quantifying the effect of these factors, except in the case of fit-up where it has been found that the effect of root gaps of 0.4 mm and greater is to increase markedly the risk of cracking (Fig. 5).

Factors used in assessing preheat levels

As indicated in this paper the setting of appropriate preheat levels is not the direct responsibility of the designer, who will generally have inadequate data on which to do it. However, the fabricator working from our specification is required to meet the requirements of Appendix E, *BS 5135*, which proposes a procedure for assessment.

The parameters used in this assessment are:

(1) The hydrogen potential of the welding process
(2) The carbon equivalent of the steel being used
(3) The combined thickness of the sections to be welded
(4) The arc energy applied during welding.

(1) Process hydrogen potentials

Process hydrogen potentials are grades A to E depending on electrodes, wires and fluxes used, as shown in Fig. 4. *Scale A* is used for consumables which give weld deposit hydrogen contents of more than 15 ml/100 g weld metal. This category applies to the rutile-coated electrodes classified as classes 2 and 3 which are generally specified for the welding of mild steel (grade 43).

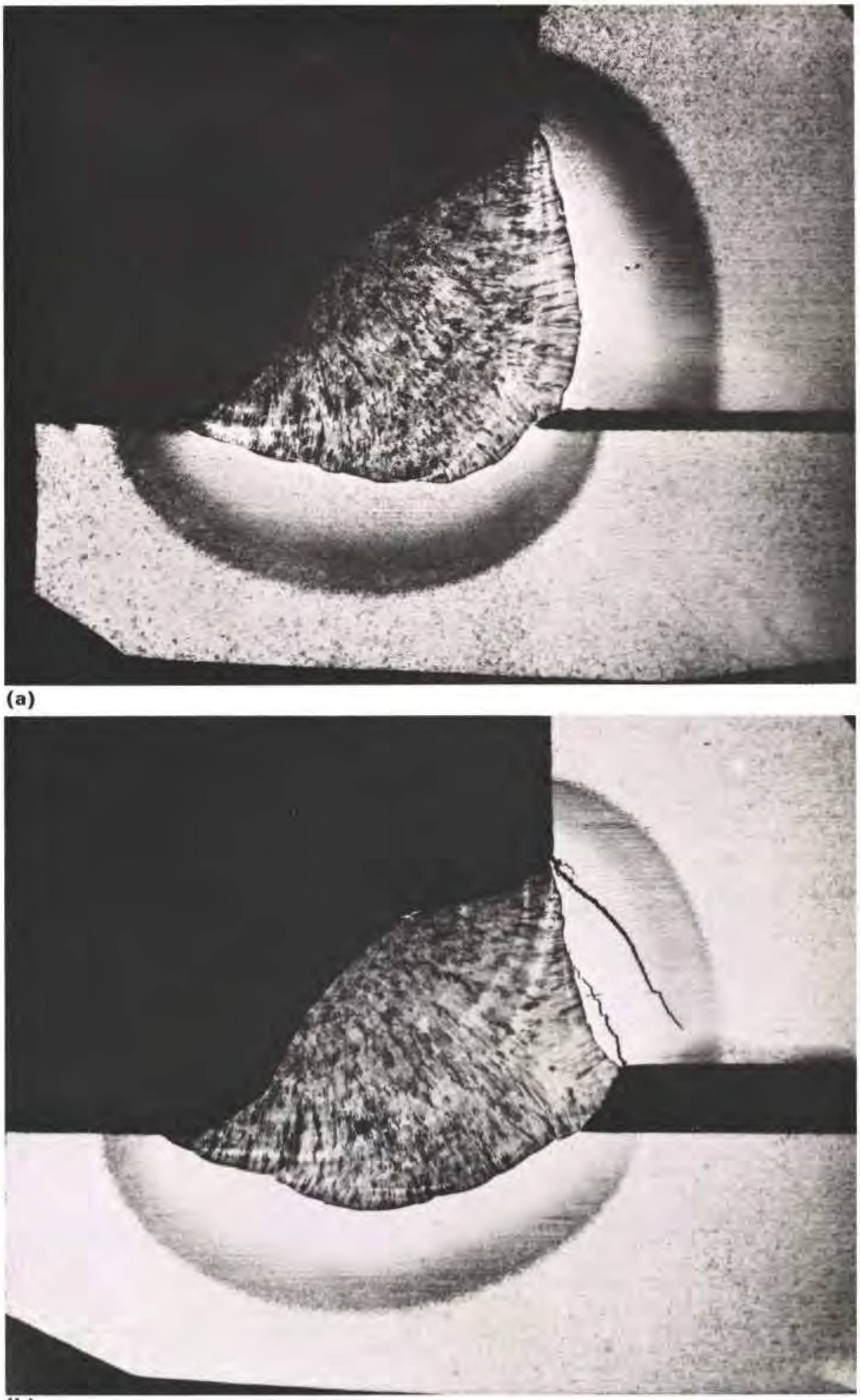
Scale B is for consumables which give weld metal hydrogen contents of between 10 and 15 ml/100 g weld metal. This category includes class 6 electrodes which are generally specified for grade 50 and grade 55 steels but sometimes also grade 43. These electrodes have a coating which absorbs moisture and they only fall in this category when dried and used in accordance with the manufacturer's instructions.

Scale C is for consumables which give weld metal hydrogen contents of between 5 and 10 ml/100 g weld metal. This includes CO_2 welding using solid wires.

Scale D is used for consumables which give weld metal hydrogen deposits of less than 5 ml/100 g weld metal. This scale is generally *not* applicable to normal structural fabrication. It can apply to specially dried class 6 electrodes or to CO_2 welding with especially clean welding wire. It also applies to welding carried out by the process using a non-consumable tungsten electrode and inert gas shielding (TIG). This process is rarely applied to structural steels but can be of value for root runs in susceptible materials. Scale D can thus be taken as applying to situations where special welding procedures are being developed to cope with particular acute problems in sensitive materials where reduced preheat levels are desirable for specific reasons.

The treatment above has ignored submerged arc welding and continuous flux-cored wire welding. When these are to be used it must be borne in mind that they can have any hydrogen content throughout the range A to D and the classification must be assessed separately for each product and application.

The scales A to D are derived from experimental relationships found between the hydrogen content which will cause cracking in heat-affected zones of measured hardness values. Broadly speaking a softer h.a.z. can tolerate more hydrogen than a harder one. Since hardness is a function of hardenability and hence of carbon equivalent, the higher the carbon equivalent the less hydrogen which can be tolerated. Preheat apart, one way of tackling this is to move down the hydrogen potential scale by changing the welding process and/or consumables.



(a)

(b)

Fig. 5
Effect of increased root gap on behaviour in susceptible material

(2) Carbon equivalents

Carbon equivalents may be calculated from the formula above using data from mill sheets which may be supplied with the steel. Whilst exact analyses will vary throughout a single ingot, and hence throughout the product rolled from it, the value will be accurate enough for the whole batch. If this information is not to hand table 1 of Appendix E of *BS 5135* presents typical values for each grade of steel to *BS 4360*.

These are as follows:

All 40 and 43 grades	carbon equivalent 0.40
50A, 50B & 50C grades	carbon equivalent 0.49
50D grade	carbon equivalent 0.43
50D1 grade	carbon equivalent 0.41

The use of these values is obviously less accurate than mill data and consequently can cause either conservative values of preheat being adopted or insufficient preheat and danger of cracking.

(3) Combined thickness

Combined thickness is the geometric factor, being the calculation of the effective heat sink which in turn determines the rate of cooling of the weld and the heat-affected zone. It is measured as the sum of the effective plate thicknesses at the joint, 'effective' being used because in the case of one of the plates being tapered, it is conventional to take the average value over a distance of 75 mm of the weld line.

(4) Arc energy

Arc energy is calculated from the relationship:
arc energy (kJ/mm) =

$$\text{arc volts} \times \frac{\text{welding current}}{\text{welding speed mm/s.}} \times 10^{-3}$$

In practice arc energy is altered by changing the welding process and welding parameters such as weld bead sizes, rod diameters and run out lengths. These directly affect current, welding speed and voltage.

The use of Appendix E of BS 5135

In Appendix E of *BS 5135* there are a series of curves relating combined thickness, arc energy and preheat level; each family of curves applying for particular combinations of carbon equivalents and hydrogen potential scales. There are curves in the standard for steels of carbon equivalents for grade A (i.e. highest potential hydrogen levels) from 0.39–0.55 which in practice covers the whole range of materials likely to be encountered. Steels within this range are classified as being of low hardenability. Above this range steels are of high hardenability and the necessary welding procedures are very different. Such steels are outside the scope of this paper.

A typical graph from Appendix E of *BS 5135* is presented in Fig. 6. Combined thickness is plotted against arc energy, the derivation of which is discussed below, and both of these are plotted for necessary preheat levels. Each graph is valid for the carbon equivalents stated in the table below for each of hydrogen potential scales A–D. The lines are dog-legged because it has been found that an increase of plate thickness above a certain limit has no effect on the cooling rate, and this thickness also depends on preheat temperature and arc energy.

One way in which this diagram may be used by a fabricator would be to:

(a) Calculate carbon equivalent from analysis, if available. Otherwise the values above may be used

(b) Assess hydrogen potential of process proposed (A to D)

(c) Select suitable graph in Appendix E from (a) and (b)

(d) Calculate combined thickness of the sections being welded as explained above

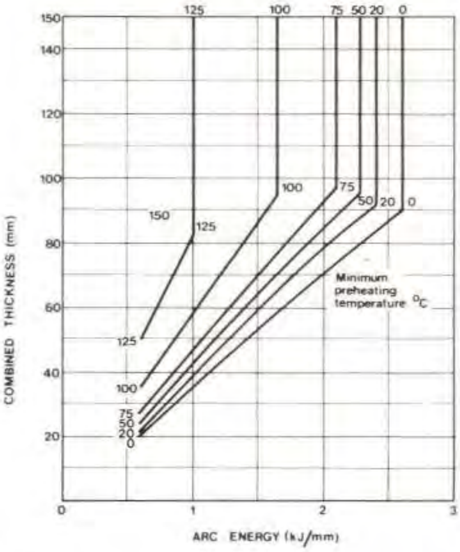
(e) Estimate size of minimum run to be used, in terms of electrode diameter and run out length of standard electrode. From this the minimum arc energy can be obtained from tables (in Appendix E)

(f) Read off minimum preheat (and interpass) temperatures from the graph using interpolation where necessary.

This procedure would be followed for multirun butt and fillet welds. However, for single-run fillet welds deposited by the manual metal arc process, a simpler procedure using tabulations alone can be followed. This is because there is a relationship for any particular class of electrode between the leg length of the fillet weld and the minimum arc energy applied. At this stage one is not required to fix electrode diameter to calculate necessary preheat, unlike the more general case of a multi-run weld considered above.

Preheating practice

Preheat is generally applied locally to the weld, as opposed to an overall heat treatment. The means of heating may be by gas torch, but this method is not reliable unless the operative is very conscientious. Electric methods using resistance finger elements, strip heaters or



	HYDROGEN POTENTIAL SCALE			
	A	B	C	D
To be used for carbon equivalent not exceeding	0·41	0·43	0·45	0·50

Fig. 6
Typical graph for preheat assessment from *BS 5135* (reproduced by permission of BSI)

blankets are preferable. Heat should be applied along the weld line and up to 75 mm either side. The correct method of checking is to test the opposite face of the steel 75 mm from the weld line to find out whether or not the intended temperature has been attained. *Tempil* sticks, which are heat sensitive crayons, are generally used, these being selected to have melting points in the appropriate temperature range. If the temperature can be measured only on the front face then one minute per 25 mm of plate thickness should be allowed for temperature equalization before measurement takes place.

If one preheats, allows the steel to cool down again and then makes the weld, there is a good chance that hydrogen-induced cold cracking will occur. If one preheats, deposits one run of a multi-run weld and then allows to cool before doing the subsequent runs, there is a chance that they may cause cracking in their heat-affected zones due to too-rapid cooling. Thus *preheat must be maintained between weld runs*. Surprisingly some welding foremen do not appear to realize this and even some inspectors and some codes of practice require root beads to be inspected before the weld is completely filled. This appears to be inviting trouble.

Use of procedure tests

It is not obligatory to assess preheat levels by using the clauses in *BS 5135*. The fabricator's experience is still acknowledged to count for a lot and a good alternative to taking either course of action is to carry out a welding procedure test. These tests are intended to be a full simulation of the welded joint. All details are written down including the materials, welding consumables (including electrode diameters), preheat, currents and voltages. Providing that the joint proves acceptable when examined both destructively and non-destructively, this procedure is then followed during the fabrication of similar joints. Welding procedure tests are generally carried out in accordance with *BS 4870: Part 1*[4]. They provide an excellent method of ensuring that good welding procedures are adopted in the first place, especially as more sensitive materials such as grades 50 and 55 steels, and have a reassurance value far in excess of their actual cost.

Conclusion

The function of preheat in the low alloy steels can be simply summarized as being to slow down the cooling to avoid hardening of the heat-affected zone of the parent metal and hence to prevent whatever hydrogen is present from causing cracking.

The procedures necessary to calculate preheat are hence apparently complicated. They are, however, a considerable step forward, enabling a workable procedure to be rapidly evolved without carrying several expensive procedure tests. In fact the clauses, graphs and tables in Appendix E of *BS 5135* are the application to a specific range of steels of various methods of avoiding hydrogen cracking valid for the majority of ferritic steels. These methods and the means of devising workable procedures are presented in Reference 3.

It may have become clear that the term preheat is slightly misleading. This is compounded by considering it as a heat treatment, since the two descriptions together imply that it is a once-for-all treatment which somehow alters the steel, rendering beneficial effects. This is not true. The metallurgical effect of preheat alone applied to the structural steels is totally reversible and it must be maintained throughout welding to be effective.

Acknowledgements

The author is grateful to the Welding Institute for providing Figs. 1 and 5 and permission to reproduce Figs. 2, 3 and 4 from Reference 3. Fig. 6 is reproduced from *BS 5135* by permission of BSI, 2 Park Street, London W1A 2BS, from whom complete copies can be obtained.

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(1) BRITISH STANDARDS INSTITUTION. BS 4360:1972. Specification for weldable structural steel. BSI, 1972.

(2) BRITISH STANDARDS INSTITUTION. BS 5135:1974. Specification for metal-arc welding of carbon manganese steels. BSI, 1974.

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Iran Museum of Modern Art

Peter Ayres

Towards the end of the '60s the Finnish architect, Alvar Aalto, was asked by Her Imperial Majesty the Shahbanou of Iran to prepare a design for a new national centre for the housing of modern painting and sculpture. Appropriately, the site chosen for this new building is at Shiraz, which is traditionally a cultural centre in southern Iran, close to the ancient capital of Persepolis. Because he was by then over 70, Aalto was not normally undertaking new work outside his native land; but, perhaps because the Shahbanou was formerly a student of architecture, and on the understanding that he would produce a design to scheme stage only, he consented to proceed.

Aalto's architecture is both personal and total. The designs he prepares for his buildings include not only the envelope and finishes, but also all furniture and fittings as well. The materials he likes expressed are the best, and natural finishes predominate – brick and stone, tiles, hardwoods, bronze and copper. In general, all elements of his buildings are clad, the structures being inferred rather than expressed.

Although he has probably undertaken designs for every kind of building, he is best known for his many major public buildings, many forming part of total town centre designs, and the Iran

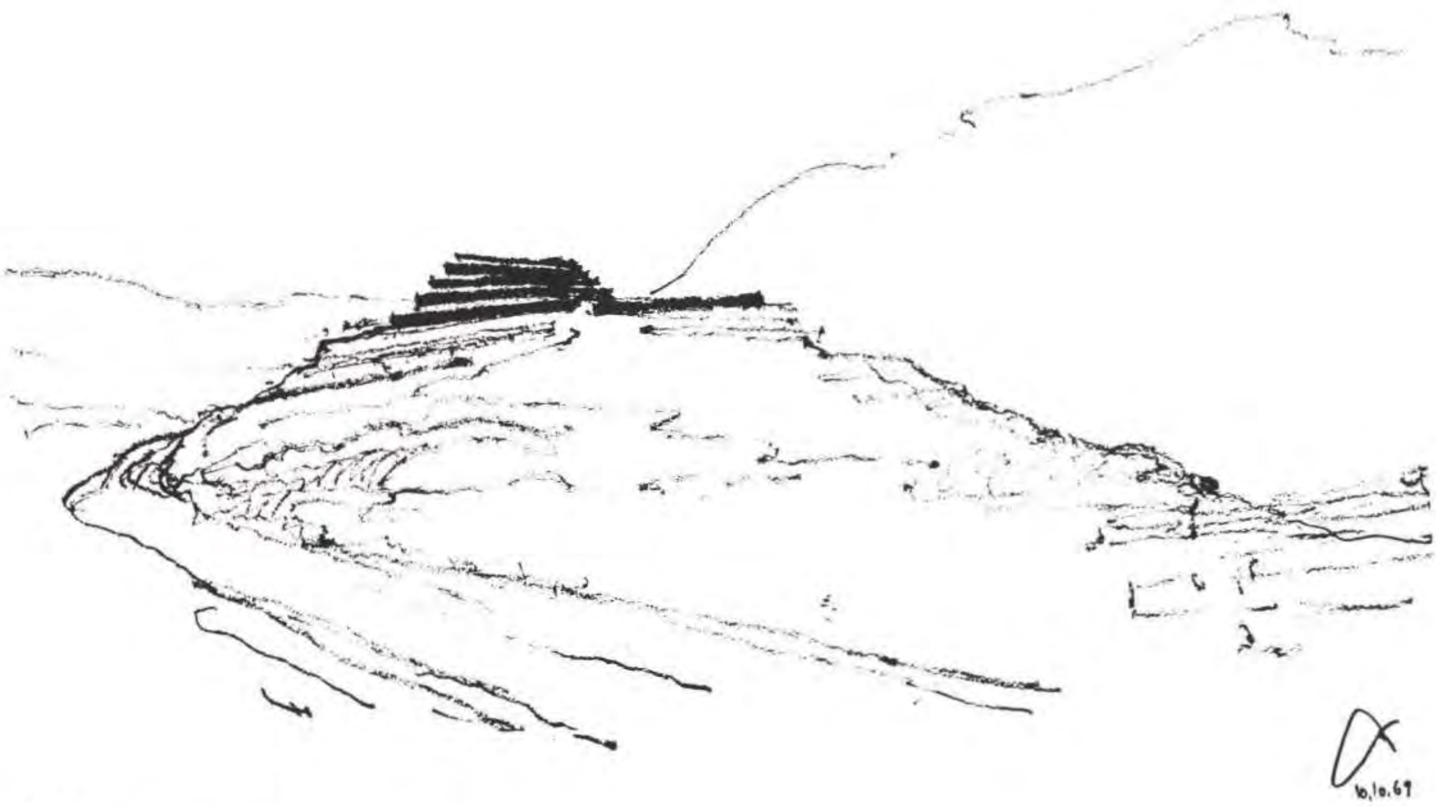


Fig. 1
Preliminary sketch made on site by Alvar Aalto

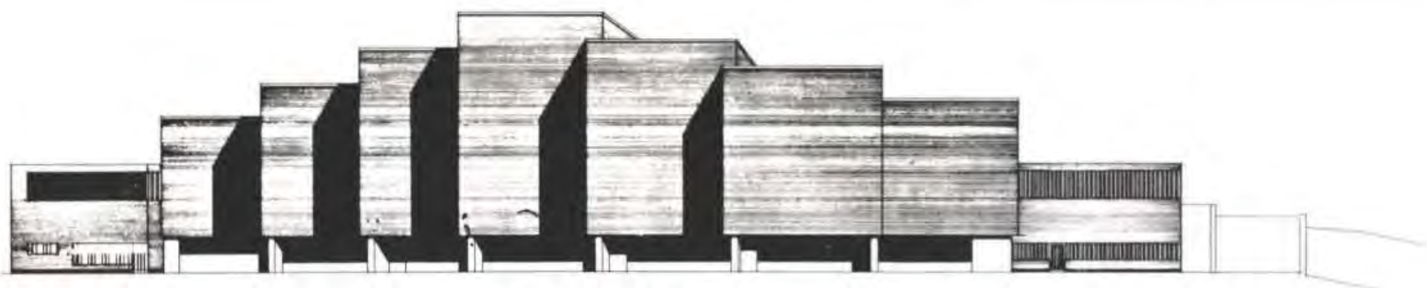


Fig. 2
West elevation showing end walls of main exhibition area

Fig. 3
South elevation showing main entrance and main exhibition area

Museum of Modern Art will fall into this category. The outstanding features which emerge from all this work are the freedom from the usual obsession with rectangular grids, and the high quality of both natural and artificial lighting normally achieved. Both these criteria occur in the Museum and the nature of the external elevations and internal spaces can best be ascertained from existing examples of his work.

The site selected for the new Museum is on top of a prominent hill, just on the outskirts of Shiraz, close to the new University complex, which affords commanding views over the whole of the city and in its turn provides a very exposed site. The surrounding area consists of a range of hills, somewhat higher than the Museum site, providing it with a backcloth of pale desert tones and Aalto's design takes full advantage of this location by providing a relatively low elevation which appears to nestle into the site. The hill itself will be landscaped and planted and will provide public walks and sculpture gardens.

The building is on two levels and occupies an area about 80m square. The plan arrangement, although carefully defined geometrically, is very irregular. One half is based on a series of radial and orthogonal grid lines, and the remainder is of almost free form, bounded by both curved and straight lines. The radial grid lines pass through an origin outside the building with an angle of 5° between each. The setting out arrangement is indicated in Fig. 4.

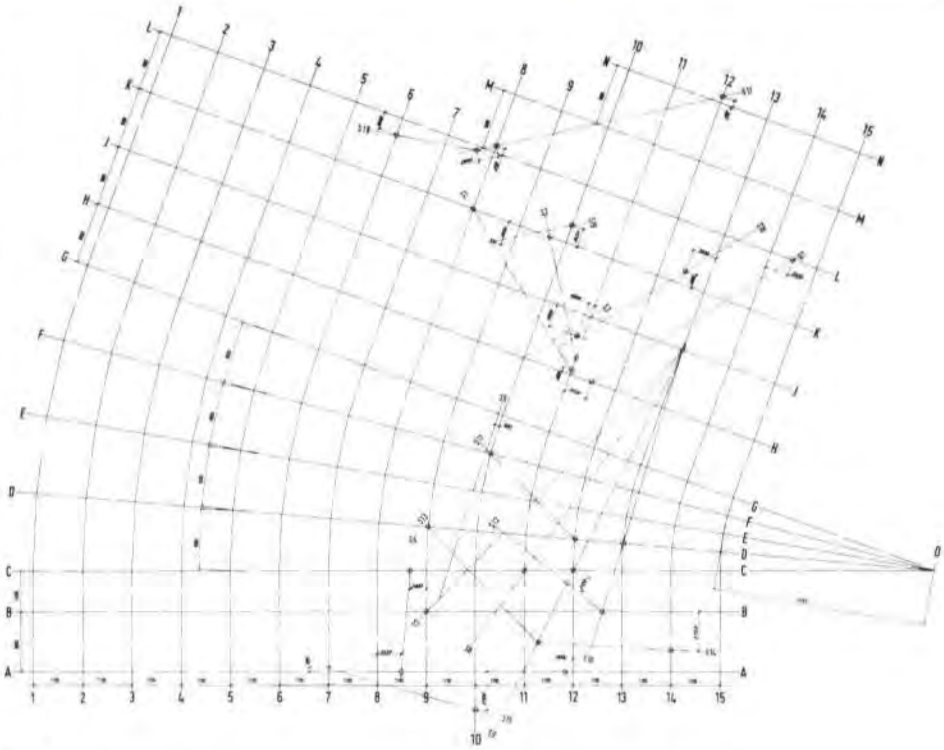


Fig. 4
Geometry of setting out

Fig. 5
Architect's plan of lower level

The structure is of in situ reinforced concrete with shallow pad footings founded on the limestone, with the floors and roofs of beam and slab construction. The difficulties of the structural design have resulted not from the system chosen but from the complex geometry of the setting out and the section profiles required, the need to design an irregular and non-symmetrical building against seismic effects and the large clear spans and unusual details required for certain areas of the roofs.

The lower level of the Museum, Fig. 5, houses the services, stores and workshops, and provides a large, covered parking area. The building is to be fully air-conditioned using a low velocity system and the large amount of equipment required is divided amongst a number of isolated plant rooms on this level. To provide adequate fresh air for the air handling plant a network of large ducts is required beneath the ground slab. To accommodate the other services, such as drainage, that are required to pass from the interior of the building to the outside, the structural ground slab is dropped below normal floor level and the difference made up by a cellular construction of dwarf walls and precast concrete slabs.

The main floor 1, Fig. 6, provides all the public areas of the Museum and its roofs the main structural and geometrical problems. The main entrance leads into a large foyer with a relatively low ceiling and clear spans of up to 20 m, lit naturally from above by a number of circular top lights. The foyer gives access to the Museum

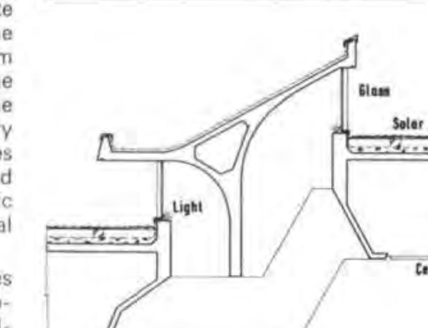


Fig. 7
Typical section through roof-light in sculpture hall

offices, restaurant and lecture theatre as well as to the two main exhibition areas.

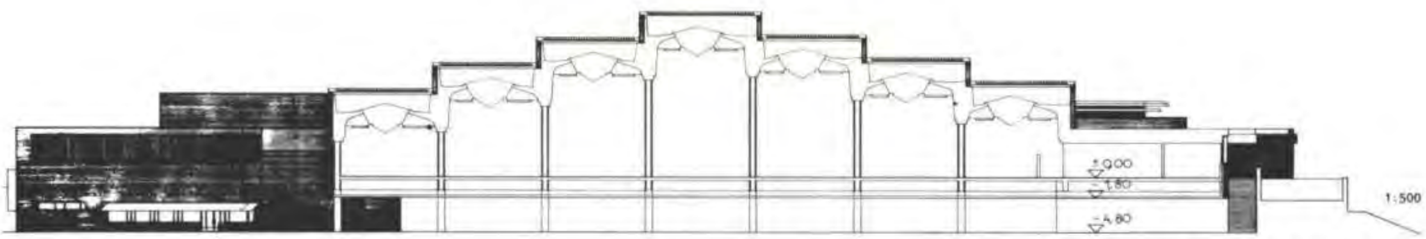
On the right hand side of the entrance foyer is the entrance to the sculpture gallery. This is a large, clear space of about 450 m² of irregular shape with a ceiling split into two levels. At the change in level is a large roof light protected from direct glare by a concrete canopy (Fig. 7).

The span of this roof varies, but at its widest point it is nearly 20 m. To achieve the required profile of the ceiling, the beams need to be cranked, and at mid-span where the roof light occurs all T-beam action is lost. One side of the room is fully glazed with doors leading out to a sculpture garden shaded by a screen of lightweight Z-sections.

The main exhibition area of about 2000 m² provides a large space that can be adapted to suit different forms of exhibition and display. There is no implied viewing sequence inherent in the planning of this area and so several exhibitions could be held simultaneously. This area is enclosed by the high brick-clad walls that dominate the external elevations and the only natural lighting comes from the roof, which is carried by slender cruciform columns with spacing ranging between 7 and 11 m. The roof itself consists of a very complex arrangement of glazing and structure. The structure consists of near rectangular cross beams, and hollow box beams running the length of the gallery. Details are shown in Fig. 8. The sections of the hollow beams vary along their length as the governing points generally lie on lines which radiate from the defining origin. Where the grid lines on plan are parallel, this change in section occurs in the vertical plane only, but where the grid lines are radial the changes occur in the horizontal plane as well. The gallery thus increases in height as one moves away from the entrance foyer, so that at the far end the walls rise to a height of over 17 m above floor level and the hollow box beams increase to a maximum depth of nearly 5 m.

Direct sunlight is prevented from entering the

Fig. 6
Architect's plan of main floor



Section of the museum tract. The roof surfaces are glazed. Slats and special reflectors guard against direct sunlight. The artificial illumination is effected from the same light source

Fig. 8
North/south section through main exhibition area

exhibition area by an external aluminium sun-screen at high level. The sealed box of double and single glazing panels provides a high level of insulation as well as the only source of natural lighting. This can be further controlled by remotely operated external shutters. From the cross beams are suspended ceiling panels which conceal artificial lighting and above the longitudinal beams run the main air-conditioning supply ducts. The return air is taken in through ducts at floor level at the base of each column.

The finishes to be used throughout the project are in keeping with the high standards to be expected from Aalto. The external elevations, including the large areas of blank wall surrounding the main exhibition area, will be clad in a dark red brick and these may have to be imported if the correct colour cannot be found in Iran. Most internal walls, columns and exposed beams will be plastered and painted and suspended ceilings will be used to conceal all ductwork. The floors will be surfaced with natural stone such as granite or marble; features like the main doors and balustrades will be clad in bronze; the window frames manufactured of hardwood and screens, louvres and similar details of aluminium.

Arups' involvement with this project began early in 1972 and a start on site was made last summer. It is expected that construction will take between two and three years to complete. Aalto's design office have not been involved in the detailed design work, which has been undertaken by architects working within the Private Secretariat of H.I.M. the Shahbanou. Being unable to conduct the normal design dialogue with Aalto has inevitably slowed the design process and the common challenge has been to maintain as near exactly as possible the forms and details proposed in the original design. If we are successful, our contributions will not be apparent in the final form of the building but rather in the fact that the Museum of Modern Arts at Shiraz will be recognized as one more addition to the long list of Alvar Aalto's great buildings.

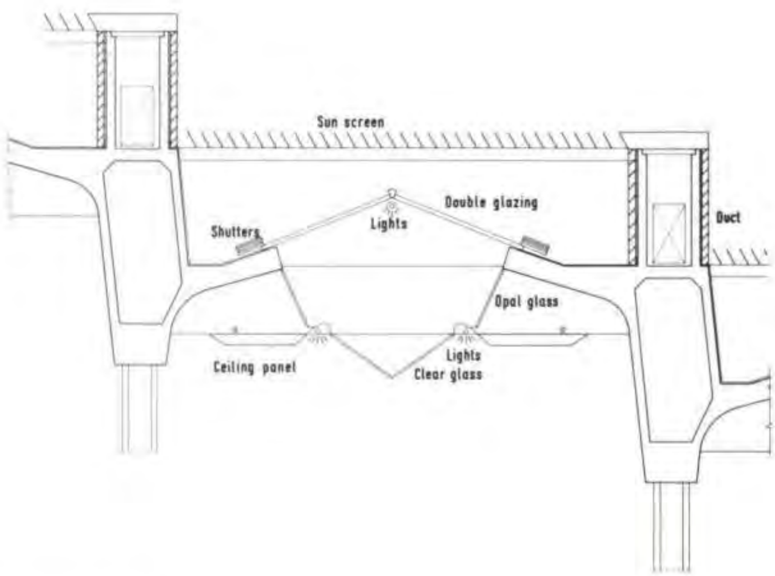


Fig. 9
Typical section through roof of main exhibition area

Credits

Architect:
Alvar Aalto
and Private Secretariat
of H.I.M. the Shahbanou

Main contractor:
Hammer Construction Co.

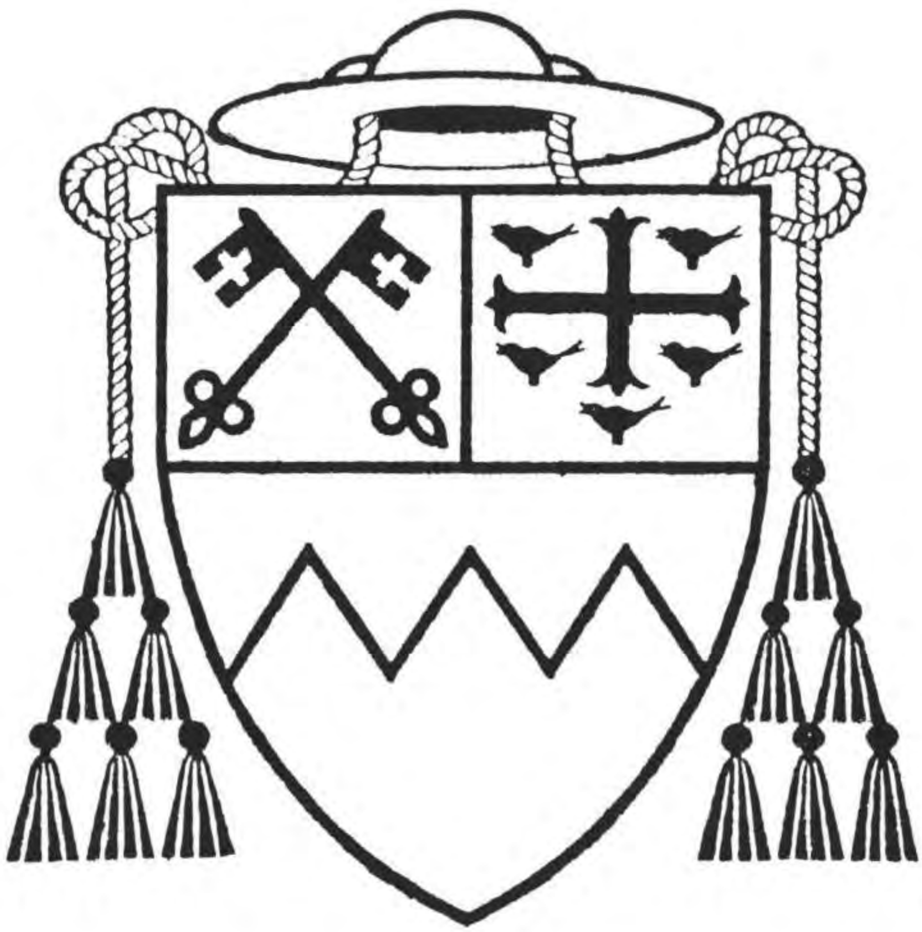
Letter

Freeman Fox & Partners
25 Victoria Street
(South block)
Westminster
London SW1H 0EX
20 October, 1975.

I glanced with great interest through the latest number of the *Arup Journal*, which you very kindly send us regularly. I was particularly interested to see the picture on the front cover – a contemporary print of the Great Exhibition Building, 1851, 'designed by Sir Joseph Paxton'. You may be interested to know that the detailed design, and the manufacture and erection of this masterpiece, was executed by Messrs. Fox, Henderson & Co. Fox was Charles Fox (knighted, along with Paxton and William Cubitt, by Queen Victoria at the opening of the Exhibition), who, six years later, in 1857, founded this firm. It is true that the concept emanated from Paxton who knew of the high capability of Fox, Henderson & Co. in light, readily erectible, cast iron and glass structures – Fox, Henderson had previously designed and built for Paxton the very large greenhouses at Chatsworth for the Duke of Devonshire, which were only recently dismantled. Paxton knew how absurdly inappropriate the Exhibition Commissioners' official and traditional design for the building was; that it could not possibly be built in the time or for less than five to ten times the money available; and he suggested to Fox that he should put in an alternative design with, if possible, a firm price attached. Fox looked into it and concluded, with the sort of layout Paxton envisaged, his firm could do the job in the very short time which would be available and for about the right money. This is indeed what happened. I believe the final price, including a number of extras, was under £100,000. The contract was only awarded late in September, 1850, and the building was virtually complete in Kensington Gardens ready for installation of exhibits by about the end of February, 1951. So far as I am aware, Paxton never stood in the position of architect as between the client and contractor. The contractor's offer was a complete design and construct one. (But Paxton certainly did a good job for the nation, and for Fox, by persuading Fox to have a go.)

After Charles Fox died in 1874, the practice was continued by two of his sons, Douglas and Francis; my father became a partner in 1912 when those two were 72 and 67 respectively. Three of their sons were in the business for short periods then a nephew, Bertram, joined my father in 1916; and these two alone continued the practice until another partner was brought in, in 1938, and my father changed the name from Sir Douglas Fox & Partners to Freeman, Fox & Partners.

Sir Ralph Freeman



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