THE ARUP JOURNAL

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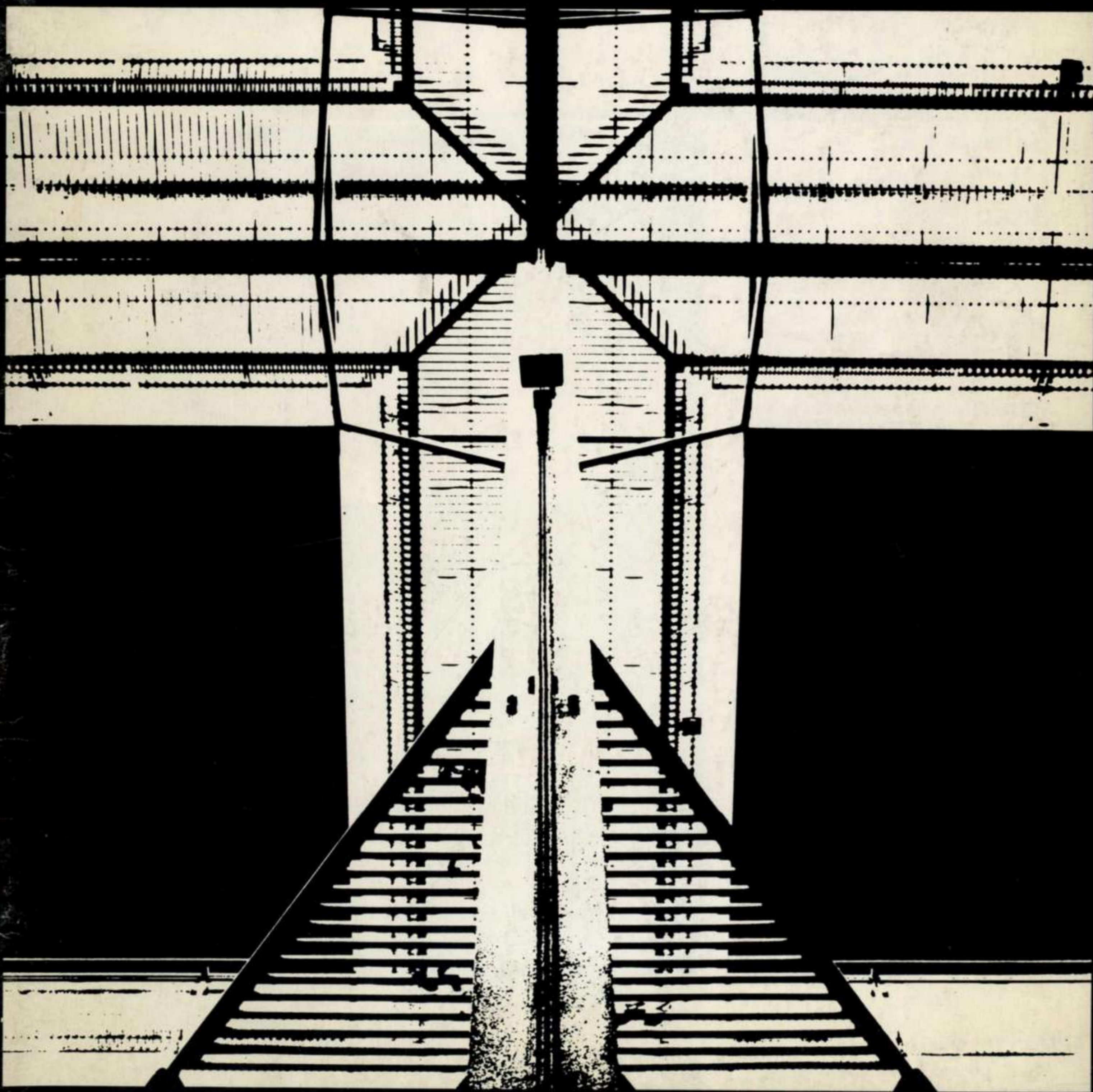

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Gateway 2

Arup Associates Group 2

Introduction

For some time since we completed Gateway House in 1976, Wiggins Teape and their property advisers Strutt & Parker had been concerned about what might be built on the empty site next door. They felt that an unsympathetic neighbour could damage not only the aspect from the building but also its investment value.

To safeguard their interests, Wiggins Teape bought an option on the site and, in 1980, we were asked to prepare a scheme which quickly became known as Gateway 2.

The brief

The brief was deceptively simple – to provide an office development which would protect and enhance the design of Gateway House.

In marked contrast to its predecessor, however, Gateway 2 was to be designed to a typical developer's brief, with 14,000m^2 of offices and a low budget. The accommodation was to be uniform width in deference to the property market, and naturally ventilated. Added to this was the Basingstoke factor – parking for no less than 400 cars.

Elsewhere in the Business Estate, this kind of unpromising brief was exemplified by several unfortunate slab-blocks, each sitting on top of a multi-storey car park, and our immediate concern was to find a way of reconciling this building type with the exuberant profile of Gateway House.

Fig. 1
Gateway 2 from the south
Gateway House is on the left

The design

A number of planning studies, supported by models, showed that the only sensible way to reduce the bulk of the new building and at the same time conceal as many cars as possible, was to plan the offices around a central courtyard. This reduced the height generally to five storeys and allowed a degree of modelling on the facade which was in scale with Gateway House. With two levels of car parking below, the main floor was approximately level with the ground at the entrance to the sloping site.

Externally, the two buildings acquired a family resemblance but, internally, they could hardly have been more different. Whereas Gateway House had an outward-looking plan, with the chief characteristic of external landscaped terraces, Gateway 2 focussed inwards to a large space, which had the potential to be glazed over to form an atrium.

The attractions of the atrium solution were obvious – a major amenity could be provided without adding to the gross area, energy consumption could be reduced because there was less external surface area and, above all, there was scope for considerable architectural drama.

The sheer size of the atrium invited the use of free-standing columns to support the roof, with pedestrian galleries spanning between at every level. These galleries greatly reduced the circulation distances around the building when connected with the enclosed stair cores, and provided the location for two groups of glass lifts.

The immediate challenge to conventional wisdom, however, was to suggest that the atrium would not add to the overall cost. This in turn meant that natural ventilation would have to work in a building 50m across. Both aspects are discussed in detail later.

Wiggins Teape as developer

The autumn of 1980 coincided with a deepening of the recession which affected all sections of industry, including paper. During the preceding year, Wiggins Teape's staff in Gateway House had reduced in number from about 800 to 550 and they had a large amount of under-used space. When the outline design for the new development was presented to Wiggins Teape in January 1981, they saw in Gateway 2 a building better suited to their current size, combined with the opportunity to generate substantial investment capital. The formula devised by Strutt and Parker enabled Wiggins Teape to take up their option on the site and become the developer/occupier of Gateway 2. The valuable freehold of Gateway House was sold to a pension fund for a sum well in excess of the development costs, providing Wiggins Teape with funds for investment in the paper industry during a time of financial stringency.

This had major design implications, such as the addition of restaurant and social facilities, a board room, a computer suite and so on. Most important of all, the programme dictated by the property transactions allowed only 18 months for construction, assuming a start on site in June 1981.

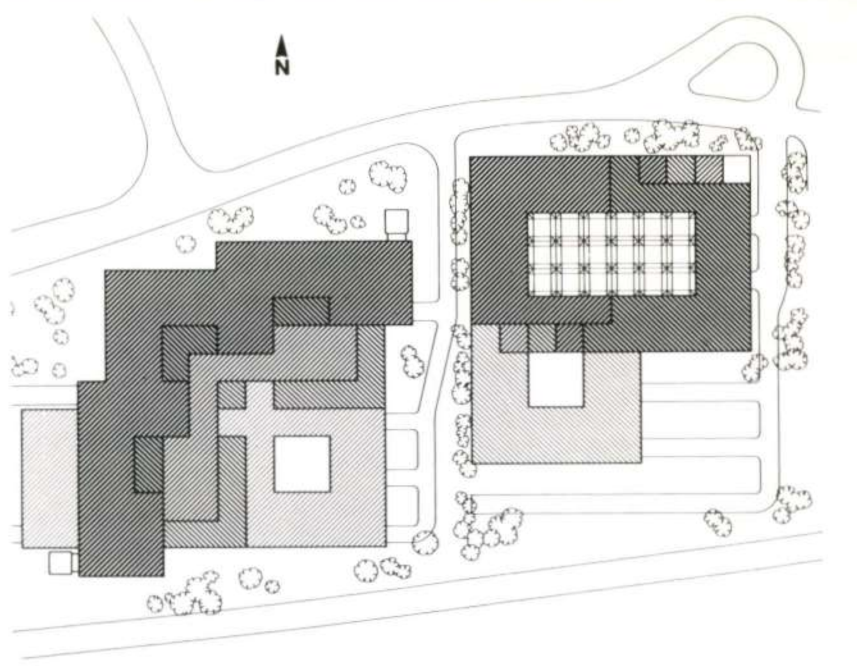


Fig. 2
Site plan: Gateway 2 on right

Fig. 3
The atrium

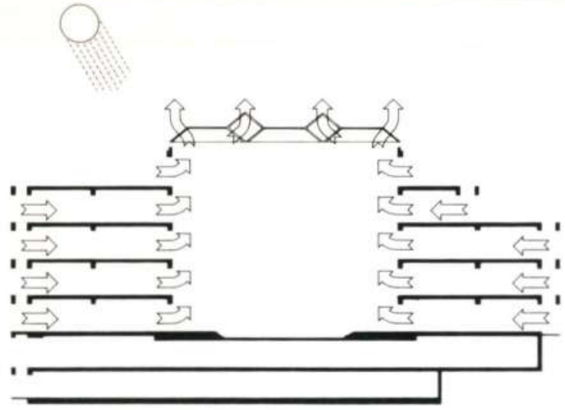


Fig. 4
Air movement: summer

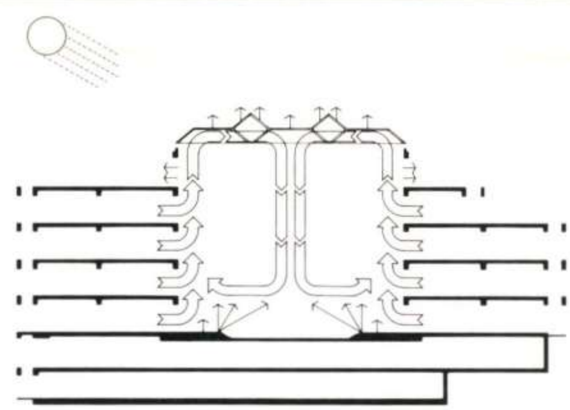


Fig. 5
Air movement: winter

Natural ventilation

The stack effect

With little time for research, a computer model developed by the Mechanical and Electrical Development Group predicted the airflow characteristics of the atrium. It confirmed that there would be sufficient height and temperature difference for air to flow naturally upwards, when the windows around the perimeter were open. Provided there were openings in the roof, the 'stack effect' would draw fresh air across the offices from outside, even on a hot still day.

It transpired that natural ventilation was not only possible but would be improved by the presence of the atrium which, by virtue of openable roof vents, would provide a degree of control not normally available in naturally ventilated buildings. Of course, individual control of the ventilation rate would be possible by opening or closing the office windows.

The cost equation

Philosophically, the low-energy, self-regulating enviroment was very attractive – an advanced building type could be made to work through the minimum use of technology – but in cost terms, the simplicity and economy of natural ventilation were fundamental. Capital and energy-consuming plant had been avoided and storey heights reduced due to the absence of ductwork, leading to a building mass which was compact and efficient. Further, the cost of the atrium roof, steelwork and glass lifts was more than offset by the savings over a traditional lightwell solution.

Detailed design of the ventilation system

The success of the natural ventilation concept depended on a number of refinements which were largely the result of the computer simulations. Firstly, the window openings facing the atrium were to increase in size on the upper floors to ensure that air would always flow from the outside to the atrium, not the reverse. Secondly, excessive heat gains in summer were to be avoided within the office space by maximum use of natural light, permanent sunshading on the exterior facade and tinted glass. Thirdly, the concrete structure was to be exposed within the offices, so that the thermal mass could help reduce extremes of temperature. Fourthly, there had to be a minimum of 130m^2 of opening vents in the roof.

The pneumatically operated roof vents were split into six zones to cater for varying weather conditions or ventilation requirements and, although all vents have been linked to the fire alarm system, they are generally controlled from the building automation system in the main plantroom.

Radiant underfloor heating was installed in the atrium to supplement the heat gained from the office space during the winter months and to keep down-draughts well above the main floor level. Elsewhere in the building, the computer suite's heat reclaim chillers provided free background heat in the atrium 24 hours a day, adding another stabilizing influence to the energy cycle. Temperature stratification in the atrium, which was to be expected in summer, has been confined to a hot layer immediately above the highest storey.

Fire safety

Clearly, the principle of natural ventilation with an atrium had the potential to affect all floors in the event of fire. In granting approvals and relaxations, the attitude of the controlling authorities was governed by the safety characteristics inherent in the basic design. The most important of these were:

(1) Gateway 2 was a simple office building and a low-to-normal risk.

(2) The atrium was restricted to circulation only and was not multi-use.

(3) Escape routes were wholly independent of the atrium.

(4) Escape distances were well inside the permitted maximum.

(5) It was not possible to enter the atrium without passing the escape stairs in the protected shafts.

(6) The protected shafts served as fire breaks in the internal corners of each floor.

In theory it was assumed that smoke would enter the atrium as if the floors were open-sided but the final window design allowed only 14m^2 of opening on to the atrium from any one compartment. This compared with 130m^2 of opening vents in the atrium roof, and a reservoir volume of 25,000m^3.

For these reasons, the additional requirements of the various authorities were restricted to smoke detection throughout the building, automatic opening vents in the atrium roof and the adoption of a single-stage alarm and evacuation procedure.

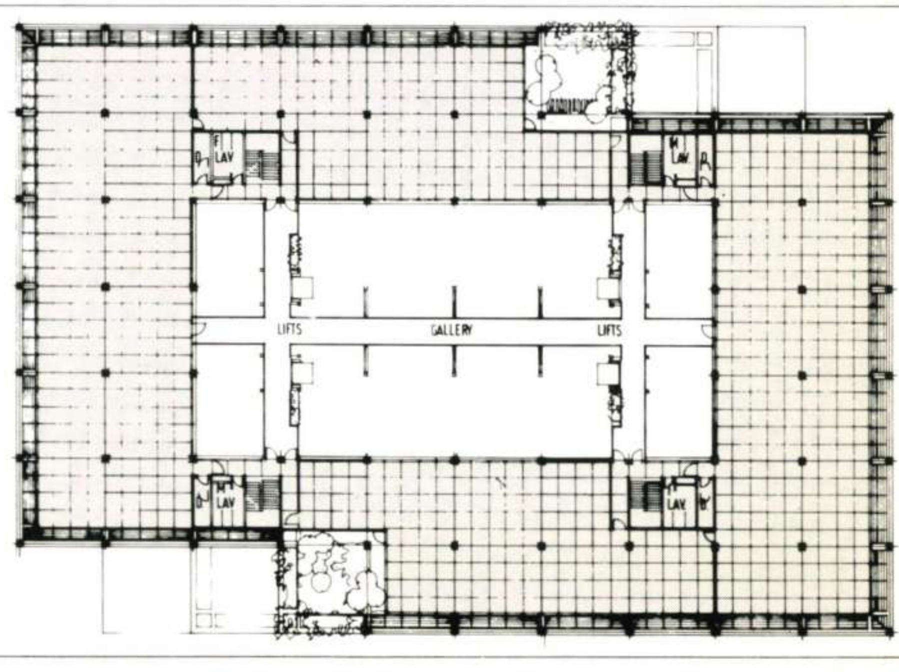


Fig. 6
Typical floor plan: level 6

Fig. 7
Erecting the external curtain walling

Fig. 8
General view during construction, taken from Gateway House

Structure

Steel

The conservatory-like atmosphere in the atrium needed slender and elegant steelwork to counterpoint the relative solidity of the concrete office structure. Aided by a relaxation of the building regulations, the 180 tonnes of unprotected steelwork included 22 cruciform columns fabricated from four 100 x 100mm angles with a continuous 20mm spacer between. Gallery edge-beams followed a similar visual theme, with two 300mm deep steel plates separated by a continuous, welded spacer. Generally, beams and columns were fully welded at works, but bolted site connections were carried out with special high-tensile sleeves fixed by socket-cap screws for visual reasons. The rolled channels and 'cat's cradles' of rectangular tubes used in the roof structure support 4.5m square panels in each bay which were raised into position fully felted and with a ceiling on the underside. The intersecting strips of glazing between the solid panels needed no additional support and were erected from the permanent sunscreen/maintenance grillage. The same preoccupation with avoiding scaffolding led to the choice of trapezoidal metal decking for the galleries, with dense concrete infill and anti-crack mesh.

Fig. 9
Detail of steel column in atrium

The hierarchy in the structural steel details was extended to small-scale elements by using square tubes for the gallery handrailing and for some of the furniture.

Concrete

Although the atrium answered many of the speculative office's traditional shortcomings, particularly with regard to spatial quality, it was felt that there was still the need to inject some interest into the working areas. In consequence, a precast concrete unit was developed which was integrated with the services and partitions and which provided a profiled structure, without a suspended ceiling.

The 100mm thick, V-shaped units were cast to two different lengths – 6.0m and 7.5m – to achieve the desired office width of 13.5m. Steel moulds produced concrete of sufficiently high quality to be exposed and painted throughout the office floors. Diaphragm ends on the V-units acted as permanent formwork for spine beams spanning between in situ, cruciform columns.

Partitions running parallel with the precast units located in the recessed junctions inbetween, eliminated the acoustic crossover problems of suspended ceilings. Those partitions which ran across the direction of the units were infilled above with triangles of glass, which fitted in grooves cast into the soffit at 1.5m intervals.

The cost of the concrete structure compared very well with the cost which might have been anticipated for a more conventional solution, which would have required a suspended ceiling and the time necessary for its installation.

Construction

During construction, general scaffolding in the office areas was avoided by means of a mobile support system, which allowed other trades to follow closely behind. Extensions of time due to bad weather were excluded from the contract conditions and the concrete contractor made extensive use of portable tents to defeat the severe winter of 1981-82, during which only one working day was lost because of the weather. Pre-glazed curtain walling was erected in storey-height units from a small hoist, which was specially designed to travel along the outside edge of the concrete structure. On the atrium side, the enclosing screens were lifted into position from the cleaning rail at roof level.

Fig. 10
View of the atrium with temporary screens to the offices and exposed metal decking to the galleries

Project organization

The contract

Wiggins Teape were already familiar with the management form of contract as a means of speeding up the design and construction process. Nevertheless, the method usually adopted for selecting the managing contractor had worried us for some while, as the successful firm could be chosen on the basis of the proposed fee and costs, rather than for their management skills and creative ability. The difficulty of persuading a client to proceed with other than the lowest tender could be out of all proportion to the ½% or 1% saving likely to be involved.

Further, the programme for Gateway 2 was so tight that the need to concentrate the contractors' attention at the selection stage on management ideas was greater than ever.

In order to effect a change of attitude, the prospective management contractors were sent an invitation document which explained that they were required to make competitive submissions based upon organization and construction methods. Cost plan allowances for construction organization and the management fee were fixed by us and given in the invitation document as two lump sums.

In addition, the invitation document broadly defined the current progress of the design, anticipated programmes for two or three of the major subcontracts and the manner of the contractor's submission. It stated that the sucessful contractor would be the one who most convincingly demonstrated that the building could be completed on time and within the authorized cost.

Five contractors were required to present personally their proposals in our Soho Square office and to leave a summary in the form of a printed formal submission. The overall quality of the presentations and the eventual choice of a highly motivated management contractor certainly contributed to the project's success, both in terms of programme and budget. The incentive to think of new ideas led in a small way to a refurbishing of the management contract, and served to reinforce the teamwork concept of client/designer/contractor.

For management contractors, this method of selection has since become a common method of competing for work.

Shared project organization

Prior to Gateway 2, our repeated use of management contracts had resulted in a very systematic approach to the way we would separate a project into its elements and distribute our information. By making use of a computer link with the site, we exploited this aspect of our organization when communicating with the contractor.

The site was equipped with a Superbrain 64QD, modem and printer to match the installation in our Soho Square office.

We identified these obvious trial areas:

(a) Instructions, drawing issues and specification approvals
(b) Monthly applications and certificates for payment
(c) Interim and final statements of account and registers for account settlements
(d) Registers for specifications and maintenance manuals
(e) Joint project reports
(f) Drawing registers.

Whilst not all these trial areas were fully implemented before completion of the project, the experiment did demonstrate that linked computers can be used to encourage closer integration and greater efficiency.

The system was particularly successful in the area of instructions, 1500 of which were issued over a range of 100 separate building elements from a computer-held library. After the addition of any specific clauses, drawing numbers and cost changes, the instructions were transmitted to the site over a telephone line and the cost information to our own DEC 10 over a hard line. By this means all instructions were typed at the site terminal on the day of issue and the cost simultaneously updated, thus ensuring a closely monitored cost control system.

Another area where there was a marked improvement in efficiency was the distribution and chasing of subcontractors' accounts – an activity made all the more important by the proliferation of subcontracted elements and the absence of the more traditional lump-sum contract. On Gateway 2, our computer system 'Intercap' collected together all the instructions and costs issued to all subcontractors and automatically produced an interim statement of account for each element every month.

Using his site machine, the contractor extracted the information via the BT network from our DEC 10 in Fitzroy Street. Chasing was then progressively carried out by the contractor from the site which, due to his daily site contacts, was more effective than when this operation was carried out by us from the Soho Square office. As a result, most subcontract accounts were settled shortly after handover.

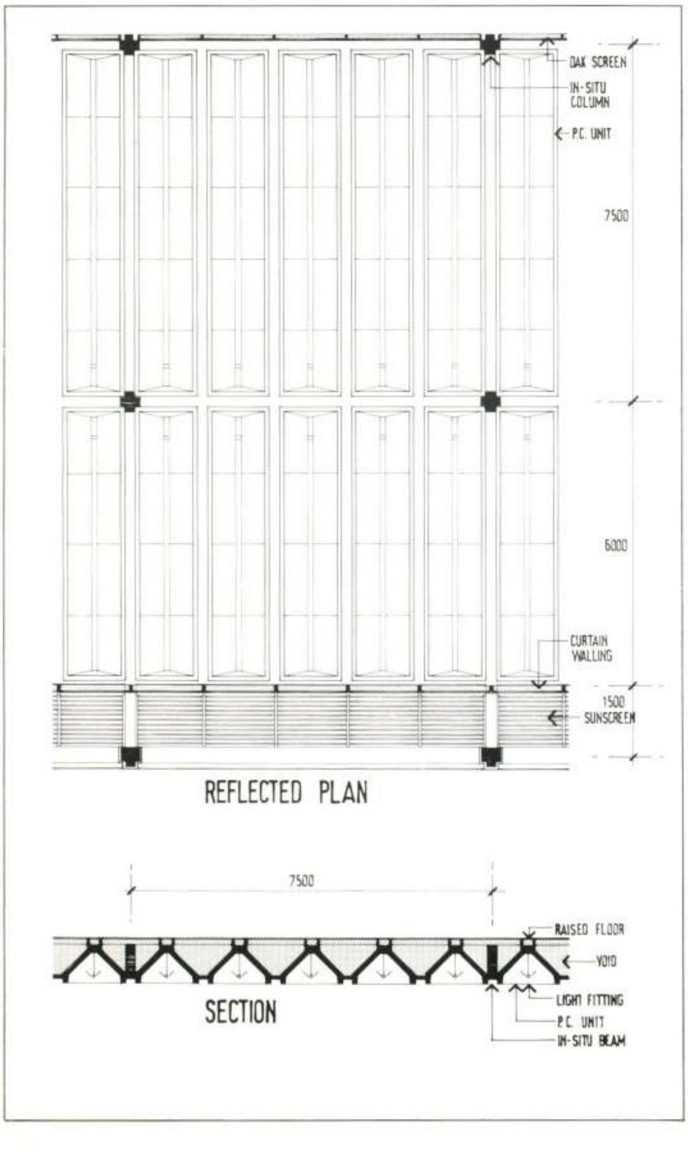


Fig. 12 Typical interior

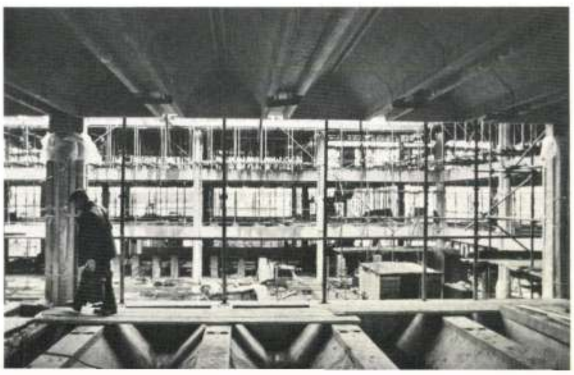


Fig. 13 Office structure without raised floor

Cost

A commentary on Gateway 2 would not be complete without some reference to the overall cost but, as the building provided a gross area of 20,000m^2 of which 6,000 was car parking, comparison with other buildings is particularly hazardous.

The budget was at the lower end of the property developers' range and the final account, at £8.7m, included £850,000 spent on fitting out, which was added without extending the contract period.

Perhaps the most important cost influence next to the design itself was that associated with time.

The choice of contract allowed the appointment of a management contractor within a few weeks of the go-ahead in January 1981, just as the detailed design was beginning. Some of the techniques for faster construction have already been touched upon, in particular those which had an influence on the design. But a great deal of effort was also put into compiling tender lists for each element, so that the lowest tenderers would not turn out to be unreliable performers.

Careful interviewing and clear presentation of information helped to convince prospective sub-contractors that the project would be well managed and that the proposed sequences for the works would be maintained. This was reflected by keen competition on all elements, in market conditions which were slowing previous inflationary trends.

In the event, adherence to the programme produced other efficiencies appreciated later, when the sub-contractors' accounts were settled without significant claims.

Suffice it to say that the project was completed on time and well within the budget.

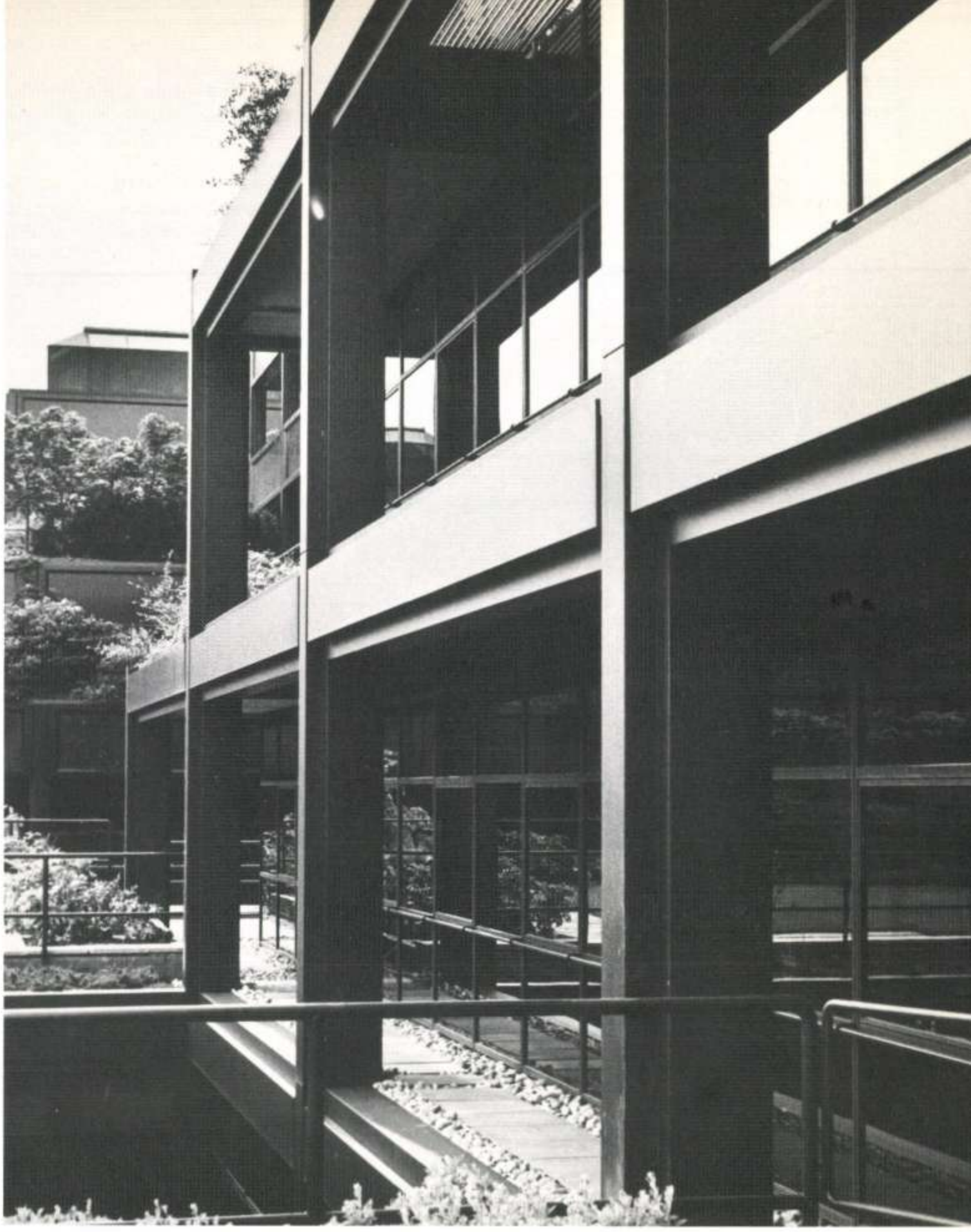


Fig. 14
Looking towards Gateway House from the roof terrace

Fig. 15
View into the atrium

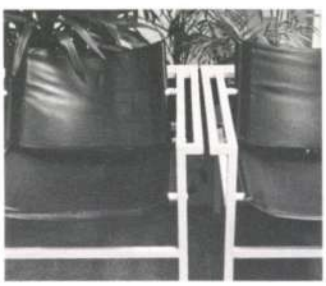
Fig. 16
The offices, seen from one of the central galleries in the atrium

Conclusion

At a technical level, Gateway 2 appears to have met with approval. Although Wiggins Teape have had to adjust from the consistency of air-conditioning to the variations of natural ventilation, there have been few teething problems other than during the heatwave last summer. Even then, temperatures inside the office were below the outside maximum by about 1½°, although it is probably fair to say that this was due to the cooling effect of the structure rather than to the ventilation rate. In winter, there appears to have been little need to open more than a few windows for fresh air, thus reducing draughts and airborne heat losses.

The atrium itself has been a great social success. Wiggins Teape's staff club uses the central area for badminton every day and, despite the limitations on its use during office hours, the atrium has been a dramatic venue for many functions in the evening. Perhaps the most imaginative was a mannequin parade, when the models descended to the main floor level in the glass lifts. During daytime, the general comings and goings and the interest provided by the moving lifts and cascading plants have led to several senior staff changing their offices, so that they can look into the atrium rather than at the view outside.

The atrium is illustrated overleaf ▷

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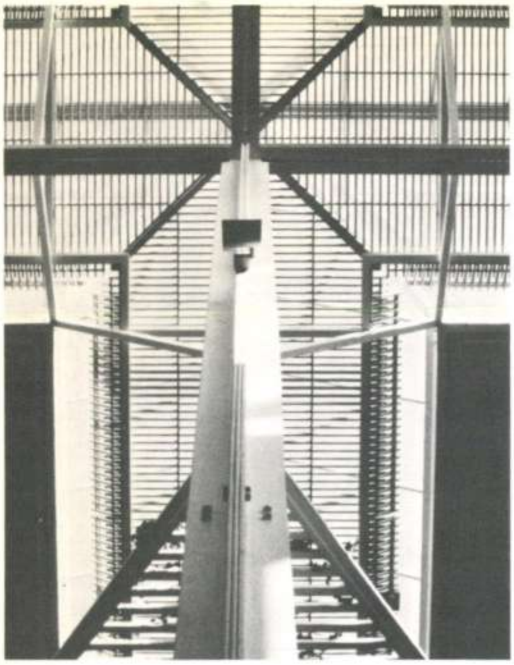
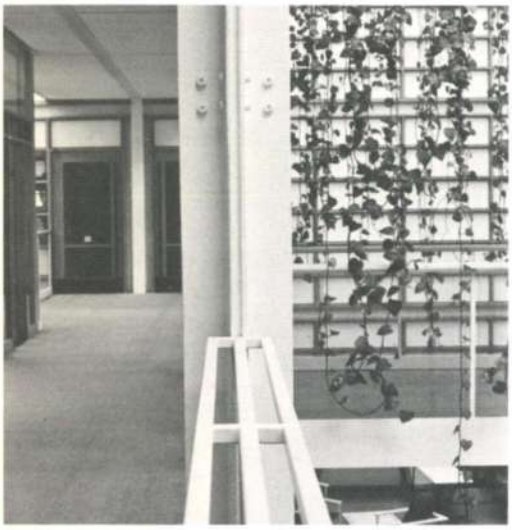
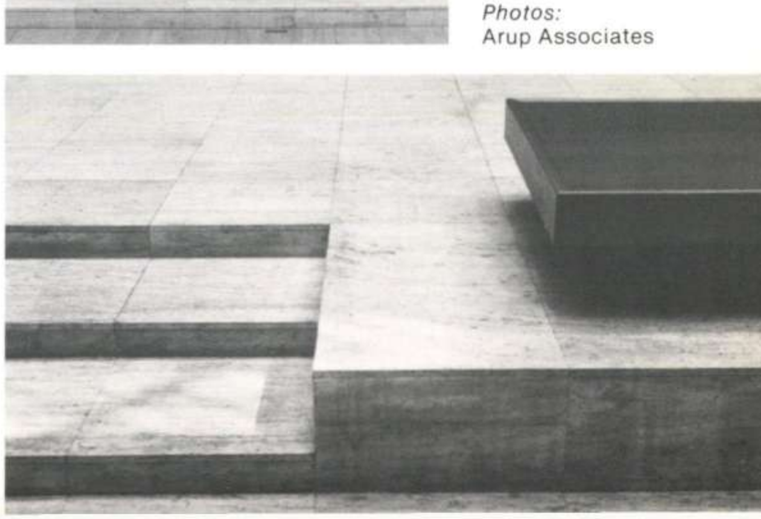
FIXED ASSETS CONTROL





Credits
Client:
Wiggins Teape Ltd.
Designers:
Arup Associates
Main contractor:
Bovis Management Contracting Ltd.
Photos:
Arup Associates



Glass reinforced cement

Michael Courtney

This paper was given at THE ARUP PARTNERSHIPS seminar 'Innovation in Practice,' November 1983.

Fig. 1
Credit Lyonnais. Architect: Whinney Mackay-Lewis Partnership (Photo: David Leech)

Introduction

Glass reinforced cement is one of the few new materials to appear in the building world in recent years. It is not and, almost certainly, never will be a major structural, or even architectural material but its development illustrates the strengths and weaknesses of the industry and the problems of innovation.

In order to use a new material, or to use an existing material in a new way, it is necessary to interpret and translate the information and knowledge of the properties of that material to a form suitable and applicable to the intended use. Research and development information tends to reflect and be limited to the interests and training of the developer or researcher.

There is virtually no original research funded or carried out by those who own, design or construct buildings. Virtually all work is undertaken by manufacturers of materials, Government research establishments or places of higher education. All of these have vested interests or have limited knowledge of real buildings so their work concentrates on particular ideas or intentions. Buildings, however, consist of materials in different forms and in different combinations with other materials and are intended to last for a considerable time.

The progress of the use of grc in buildings demonstrates this same conflict and problem.

Initial development in grc was very slow and cautious, concentrating on isolated properties of a single form of the material, spray dewatered grc. Pressures of commercial exploitation however meant that manufacturers started with a different material, direct spray grc containing sand filler, and have moved even further away by using grc in combination with other materials, styropore and polystyrene, to form sandwich panels. Problems have arisen now with grc for which it has been more difficult to establish cause, effect and remedy, due to the material's change of properties with time.

The following consideration of this progress is necessarily simplified and condensed.

The material

Glass reinforced cement is a combination of glass fibres, cement and water with sand filler and admixtures. It can be made in three different ways, each of which produces material with different short and long-term properties. Spray dewatered material is made by using a high water content and then compacting and dewatering by suction. Direct spray material has a low water content, uses an additive to achieve workability and is compacted by rolling. In both these techniques glass fibre and cement slurry are sprayed into moulds and only combine at the point of contact. Premix material is similar in consistency to direct spray except that it is premixed, is vibrated rather than rolled and has much lower qualities.

The idea of reinforcing cement mortar with glass fibres to make a homogeneous ductile material with tensile strength has been in

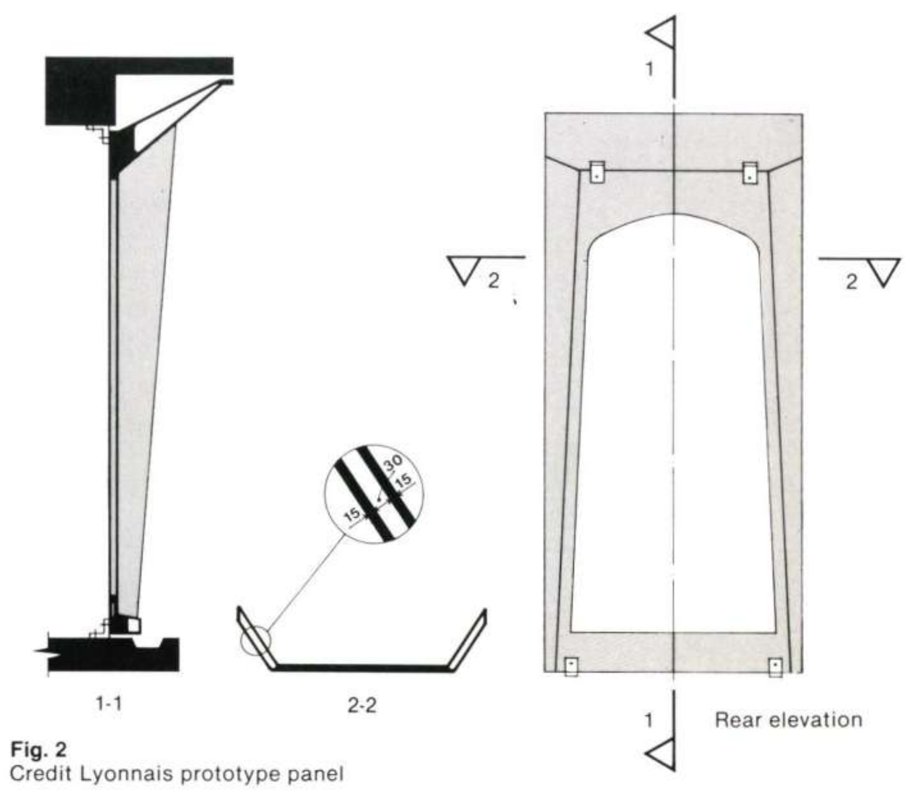


Fig. 2
Credit Lyonnais prototype panel

existence for some time. Normal glass is however attacked by the alkali in cement and the qualities of the composite are lost. In 1966 a breakthrough was achieved at the Building Research Establishment when Dr. Majumdar discovered that zirconium-rich glass had a resistance to alkali attack, was able to make fibres of this glass and to combine them with a cement paste matrix. The material was patented through the National Research and Development Corporation and preparations made for commercial exploitation.

Building Research Establishment

The development of glass reinforced cement at the BRE occurred towards the end of the period when problems with high alumina cement had created great difficulties. The Establishment determined that glass reinforced cement should be subjected to a lengthy test programme before they would endorse its commercial use.

The test programme was established by the scientists at BRE and was therefore scientific in concept. The programme concentrated on one material, kept in controlled environments and tested in a way that was repeatable and gave consistently reproducible results. (Where work is based on test results and experiments which can be reproduced by others it can be published.)

Initial work had shown that the alkali attack was resisted, not prevented, so the material's properties changed with time; it aged. This process occurred much faster in water than in air so the two carefully controllable environments chosen were warm water and dry, cool air. The initial work had also shown that the material became brittle and was difficult to test in direct tension, as any distortion of shape or test grip alignment introduces stress concentrations. The test chosen to monitor the quality of the material and its change of property with time was, therefore, the four-point bending test on spray dewatered grc.

None of this work is directly applicable to the commercial use of direct spray grc in the building industry.

It was expected that the results of the ageing process would be clearly demonstrated within five years, that the effects of all environments would lie between those of dry air and those of warm water and that a relationship could be established between four-point bend modulus of rupture (MOR) and direct tension (UTS). A small number of samples were however exposed to natural weathering at the BRE station, Garston, and some tensile testing was planned.

Pilkington Brothers Ltd.

Pilkingtons were granted a licence by NRDC for the commercial production and exploitation of the alkali resistant (AR) glass.

Having proved that the fibre could be produced commercially and having built a pilot plant, Pilkingtons were keenly aware that the building industry would be very cautious. Many were still suffering from the problems with high alumina cement which also has properties which change with time, and with the performance of small unskilled firms producing glass fibre reinforced plastic materials.

Pilkingtons wished to sell glass fibre, not grc; however, to develop a market despite these two problems, they decided to establish an extensive research programme to provide much more information to users, to provide an extensive technical support service and to establish a licensing system for grc manufacturers to whom and only to whom they would sell AR glass fibre marketed as *Cemfil*. With a material which has good short-term but poor long-term properties, inadequacies in manufacture can be a particularly difficult technical and contractual problem.

By the early to mid-1970s, however, *Cemfil* Marketing Division of Pilkingtons was experiencing intense pressure for the commercial exploitation of grc in order to develop a viable market for Pilkingtons' product and to recoup some of the development costs. Yet already decisions relating to product manufacture and commercial investment had changed the material which was being promoted from spray dewatered, neat cement paste grc to direct spray, cement paste with sand filler grc. To satisfy the need for rapid answers on this material, Pilkingtons had become committed to accelerated ageing by immersion in hot water to provide samples for their testing programme.

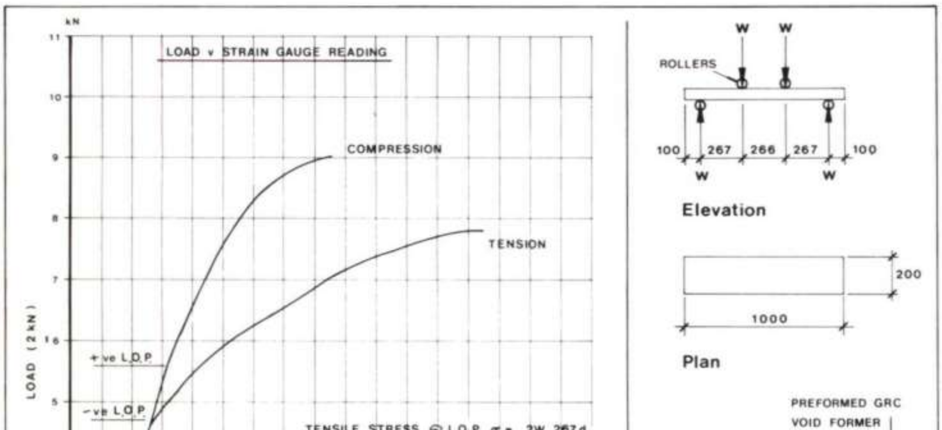


Fig. 3
Box beam model tests for Credit Lyonnais

Initial uses of grc

The initial use in structures was limited to permanent formwork, where the early qualities of the material are used. Even in this, however, problems were experienced in the lack of understanding in design that a stiff material acts differently from the more conventional ductile materials.

In terms of product promotion architectural cladding makes the most impact in the building industry even though it is a relatively small volume market. Some work had been done using grc as cladding and in 1974 Ove Arup and Partners received a commission to advise on the structural aspects of the use of grc as cladding to the new London branch of Credit Lyonnais (Fig. 1).

Credit Lyonnais

The published information on grc was gathered and studied, intensive meetings held with the BRE, Pilkingtons and the Architect and a number of critical decisions taken, all in parallel with the development of the architectural design concept and its relation to the structure, finishes and services of the building.

The decisions were aimed at reducing as much as possible the risks inherent in the use of a new material and the limitations of the information available. The material to be used would be as close as possible to that on which research had been carried out, a limit state design approach would be used to relate loads and resistance in order to encourage consideration of different factors, the characteristic tensile strength would be based on the elastic linear limit of the material, the material would be used in a manner that it acted on its own and not as a combination with other materials, tests would be carried out on models and prototypes to prove structural analysis conceptions and intensive supervision of manufacture would be undertaken.

A design was prepared on this basis (Fig. 2) and the programme of tests on models, fixings and a single prototype panel undertaken. The fixing tests gave very high results but the box beam model tests (Fig. 3) demonstrated that the structural mode was direct tension in one face and not flexure, so the skins had to be thickened to accommodate this lower limit. The full-scale prototype panel demonstrated a load-carrying capacity of more than 10 times the design load, so everyone was happy. The design has apparently been successful and there are no problems so far with the cladding.

Research and material properties

In 1975 BRE collated the results of the first five years test programme on grc and prepared a prediction of the value of material properties at 20 years. The results caused considerable discussion before being agreed between Pilkingtons and BRE and published[1].

The primary problem arose regarding the predicted value of ultimate tensile strength of the material exposed to natural weathering. It had become apparent that this was the property which would control most of the architectural designs and uses in the building industry and was proving most difficult to agree and predict.

The behaviour of material exposed to natural weathering did not seem to lie proportionately between that of the two controlled environments and had not yet reached a steady state. A detailed study of individual test results and problems of prediction by curve fitting indicated that the shortage of relevant test specimens was leading to undue weight being given to certain low results. Although these probably indicated poor specimens or poor testing the lack of available data meant they could not be neglected.

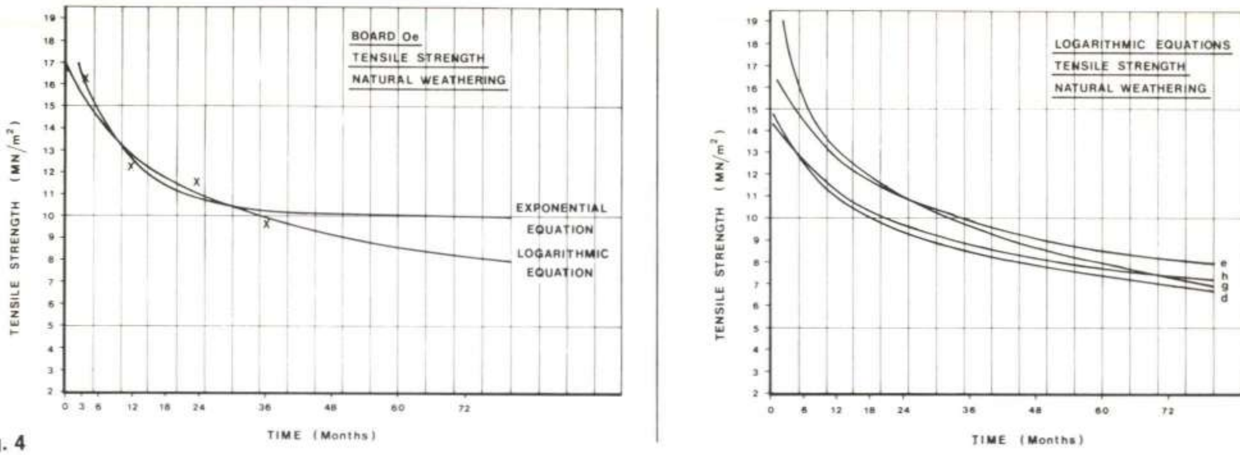


Fig. 4
Tensile strength with time

Pilkingtons commissioned Ove Arup and Partners to provide a study of material properties and suggest a design approach. This study[2] brought out the difficulty derived from low test results (Fig. 4) and suggested a design approach based on the published table of predicted 20-year properties (Fig. 5).

Design guidance and market development

Pilkingtons found that, while the market for grc was expanding appreciably, they were still giving free design advice and this was becoming a very heavy burden on their resources. To assist in promoting the material, while lessening their involvement in individual project design, they published a guide to the use, design and manufacture of grc[3].

The market for the material became very much wider than the initial architectural cladding use and manufacturers started producing pipes, roof tiles and asbestos cement replacements among many other products. The use in cladding also changed. With its adoption by more designers who had little or no knowledge of its qualities and properties, most of the work came to be carried out by manufacturers on a design and construct basis at the same time as Pilkingtons were withdrawing from their role as quality control insurers and concentrating on fibre production and sales.

The very competitive tendering climate in the worldwide building industry, and particularly in Britain, and the desire to increase the market penetration of grc, led manufacturers to change their methods. They adopted less rigorous standards of quality control, used material which, by the incorporation of higher quantities of cheap sand filler, was less like the material on which data was available, and they combined it with other materials to form sandwich panels which were easier to make and provided additional properties within the building envelope.

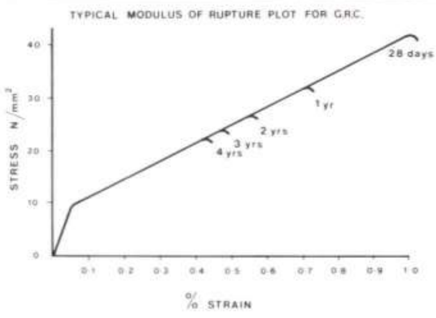
Formality

In 1978 the BRE published the results of the 10 year tests on their grc samples[4]. These confirmed the work done after the five year tests. Little comment was generated in the industry as virtually no grc of this type was being produced.

Nemesis

During the 1970s the market for the use of grc continued to expand throughout the world. There were occasional reports of cracks in grc cladding panels. These problems were found to be associated with poor design, poor workmanship or poor quality material and these were accepted as causes.

In late 1981 reports began to circulate through the industry of serious and extensive cracking in grc cladding panels and in 1982 Pilkingtons produced a limited circulation report confirming that they had discovered an hitherto unforeseen problem with moisture movement strains in curved and shaped sandwich panels[5]. Differential moisture movement drying shrinkage, between the outside skin and the inside skin of a shaped sandwich panel, could be enough to generate moments in the panel, from the restraint of its shape, sufficient to cause cracks. Fig. 6 shows a typical shaped panel.



Property		Air	Water	Weather
(a) Bending				
MOR	(MN/m²)	26—34	15—20	13—17
LOP	(MN/m²)	14—16	15—18	13—16
(b) Tensile				
UTS	(MN/m²)	11—15	6—8	5—7
BOP	(MN/m²)	7—8	6—8	5—7
Young's modulus	(GN/m²)	25—33	28—34	25—32
Impact strength	(Nmm/mm²)	14—20	2—3	2—4

Revised estimated mean strength properties of spray dewatered OPC/GRC (5wt % glass fibre) after 20 years

Fig. 5
Predicted properties GRC at 20 years (BRE)

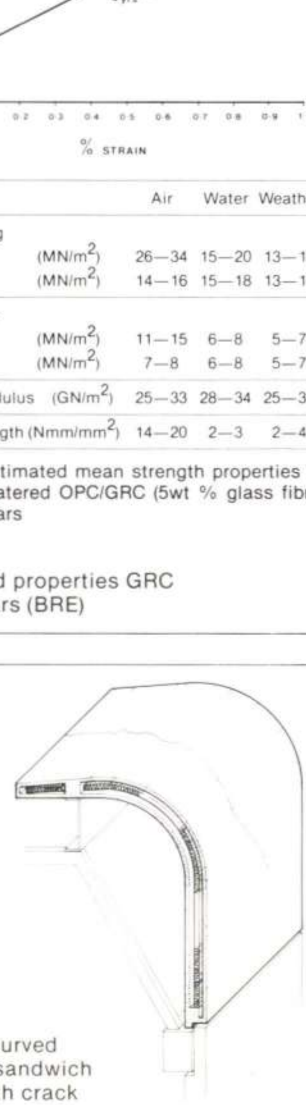


Fig. 6
Typical curved parapet sandwich panel with crack

The difficulty with this hypothesis, developed from a theoretical study, was that, whereas it predicted cracks would occur in all shaped sandwich panels, cracks were actually only occurring in some. Furthermore cracks were also occurring in flat sandwich panels and the hypothesis did not explain these.

Ove Arup and Partners were however beginning to investigate cracks in grc panels both of their own design and designed by others. Work has therefore been carried out involving field measurements, checks and controls which has demonstrated that the initial postulation of the cracking hypothesis does occur; the laboratory work has been shown to represent what can occur. What is still unknown is why it should occur in some situations and not in other, apparently similar, situations. It may be that there is some further relationship involving material quality, material combination and exposure aspect but insufficient research work has been carried out to determine these matters.

Hopefully the research work will eventually be undertaken but at present the other notorious aspect of innovation, fear of legal liability, is well to the fore and much information is being kept confidential.

Conclusions

Work in innovation is more taxing and more difficult than normal design work. It is however very rewarding as it not only needs but also encourages a wider understanding and awareness of the physical behaviour of materials and the interaction of effects.

The work in grc however shows how difficult it is to be consistently right even when proceeding carefully and evaluating the reasons for and results of extensive research programmes. The innovation work in grc has been brought about by, yet has suffered from, commercial pressures. However the present problems have arisen, not from errors in what was explicitly considered, but from a factor that was not explicitly considered. This can happen in 'normal' design as well as in innovative work.

APPENDIX A

Bibliography

(1) BUILDING RESEARCH ESTABLISHMENT. *Current paper 38/76.* A study of the properties of Cem-Fil/OPC composites. BRE, 1976.
(2) ARUP, OVE & PARTNERS. Glass reinforced cement: an appraisal of current information with recommendations for a structural design approach. OAP, 1975.
(3) PILKINGTON BROS. LTD. Cem-Fil GRC design guide. Pilkington Bros. No date.
(4) BUILDING RESEARCH ESTABLISHMENT. *Information paper IP36/79.* Properties of GRC: ten-year results. BRE, 1979.
(5) CEM—FIL INFORMATION BULLETIN 44. Pilkington Bros., 1982.

Assessment of seismic hazard

Edmund Booth

Synopsis
This paper gives some background information on the nature of seismic hazard, and describes the principles involved in selecting earthquake ground motions for use in the earthquake-resistant design of structures. It recommends how these motions can be related to the zoning in seismic codes.

Appendix A gives some statistical information on the extreme value distribution of earthquake accelerations and Appendix B gives standard formulae, based on Cornell[15], for quantifying earthquake acceleration return periods. Appendix C suggests a method for quantifying the effect of large magnitude, distant earthquakes on long period structures.

The underlying philosophy of this article is the same as in David Croft's earlier paper[23] on the seismicity of Iran, but the scope has been broadened and recent developments have been noted.

The nature of seismic hazard

The seismic hazard at a site is a measure of how likely the site is to be affected by damaging earthquakes. However, the risk of damage to a given structure depends not only on the seismic hazard at the site but also on the vulnerability of the structure to earthquake damage. This can be expressed by the relationship

Risk = Hazard x Vulnerability

We are concerned here with defining the hazard of a site, so that the vulnerability of the structures that are built there can be adjusted to give an overall level of risk that is judged acceptable. The acceptable risk level will depend on the type of structure; it will be different for low-rise housing structures than for high risk facilities (e.g. nuclear power stations), or buildings like hospitals needed for a post-earthquake emergency. Many codes[1,2,3] recognize these factors explicitly.

While the principles of selecting design forces are in many cases similar to those involved in wind engineering, there are some important differences between the characteristics of wind and earthquake loading which affect the engineering approach to hazard assessment, as noted below.

a) The ratio of loading with a long return period (say 1,000 years) to a short period (say 50 years) is much greater for earthquakes than for wind. Appendix A provides some statistical comparisons. The corollary is that the risks associated with earthquakes increase much more rapidly with return period than for wind. Since the overall risk of collapse is the product of vulnerability and hazard summed over all return periods, a structure designed to withstand a 50 year return earthquake with the same factor of safety as a 50 year wind has a much higher overall risk of failure. Appendix A investigates this further.

b) The destructive effects of a major earthquake are likely to be more comprehensive than those of a major wind storm, affecting not only structures, but also buried services (e.g. water, gas), mechanical and electrical equipment (e.g. telecommunications equipment, hospital support systems), and road and rail links. The earthquake may also trigger landslides or tidal waves. The ability to mount rescue operations and to fight secondary disasters (especially fires) may therefore well be much more adversely affected by a destructive earthquake than by a rare windstorm.

c) In general, the damage caused by a freak wind storm extends over a period of many minutes and meteorological prediction methods are well developed. However, the period of strong shaking during a major earthquake typically lasts a minute or less, leaving little time for evasive action, and earthquake prediction methods are as yet very unreliable.

Earthquake loadings with a long return period are thus not only highly destructive and comprehensive in effect, but also difficult to take evasive action against in the short term. Therefore, earthquake-resistant design has to consider events with a much longer return period than is normal for wind-resistant design. Because the events are rare, it is not considered economic to prevent damage from occurring. Instead, the design should ensure that structures do not collapse, escape routes are unblocked and services vital during the post-earthquake period can survive the shaking in a functional state.

The UBC[1] of the USA specifically excludes consideration of return period from its hazard assessment, which it bases solely on the maximum historically recorded earthquake damage at a site. More recent US codes[2,3] define the hazard in terms of a 500-year return period event, and this is recommended as appropriate for building structures. Design of high risk facilities like nuclear power stations involves consideration of even longer return periods.

Causes of earthquake damage

Earthquake damage arises from a number of different causes, as follows

(1) Dynamic effects due to ground shaking (Fig. 1)

(2) Foundation settlements and movements

(3) Permanent relative movements across a fault break

(4) Landslides triggered by the earthquake (Fig. 2)

(5) Liquefaction, a phenomenon occurring in saturated granular soils which may lose strength dramatically under cyclical loading (Fig. 3)

(6) Tsunamis (tidal waves) and seiches (changes in water level in lakes)

(7) Other secondary phenomena, for example fire and flood damage following dam failure.

Items 2 to 5 require specialist geotechnical advice and, together with Items 6 and 7, are beyond the scope of this article which is concerned with describing the ground motions causing Item 1. Historically, ground motion has caused the greatest amount of damage, though in some major earthquakes fire damage has been equally significant.

1


Fig.1
Strong motion damage (Italian earthquake, 1980) (Photo: R. Spence)

Fig.2
Landslide damage (Alaskan earthquake 1964)

Fig.3
Liquefaction damage (Niigata earthquake, Japan 1964)

2


3


Description of earthquake motions

Earthquakes typically originate at depths between 5km and 200km below the earth's surface. They are believed to result from a failure of rock in the earth's crust at weak points, under the action of high strains. The failure results in a sudden movement, felt as an earthquake. Much of the strain energy released is dissipated as heat; a residual 10% appears as seismic waves which produce the ground shaking. Accelerations up to 1.2g and permanent ground displacements of up to a metre or more have been recorded in major earthquakes.

Fig. 4 shows a record of the acceleration recorded at a particular point during the 1971 San Fernando earthquake. The damaging power of the shaking depends on three properties of the motion:

(1) The maximum ground acceleration
(2) The frequency content
(3) The duration of shaking.

These are now discussed in turn.

Methods of assessing the *maximum ground acceleration* at a site – corresponding to factor 1 – are described in a later section.

The frequency content – factor 2 – is expressed in a simple but effective way by the response spectrum of the motion, which is described in greater detail in a later section. Clearly the match between a structure's frequency and the predominant forcing frequency of an earthquake is important in governing the way the structure responds. The magnitude and depth of an earthquake, the distance of a site from the earthquake source, and the nature of the soil deposits through which the seismic waves pass, all have an important influence on ground motion frequency content.

The duration of shaking affects the number of stress reversals that a structure experiences and the amount of earthquake energy it has to absorb. Duration is therefore important in a structure's ability to survive the earthquake. Building codes specify special detailing requirements and material specifications which are found to have good low cycle, high amplitude fatigue characteristics. Explicit allowance for duration effects requires sophisticated analytical techniques inappropriate to most building structures, and its consideration is beyond the scope of this article.

Preliminary quantification of seismic hazard

A preliminary indication of the seismic hazard at a site can be obtained by referring to general earthquake zoning maps, for example Refs. 4-6 for worldwide data, Refs. 7-9 for the Middle East.

The next stage is to find out if a local code with reliable zoning exists, for example by referring to the World List[10].

If the local code does not exist or is considered unsatisfactory and if no other reliable source of information exists, a special study of seismicity should be carried out. The data sources on which this should be based are described in the next section and the methods of processing these data are described in the subsequent sections.

Data sources for special studies

There are three types of data for determining the seismic hazard of a site, all of which must be considered in a well-founded study.

They are:

(a) geological information
(b) instrumental records
(c) eye-witness reports of earthquake effects.

These are now discussed in turn.

Geological information includes the overall tectonic and geological setting of the site, and also more local factors, including the proximity of potentially active faults, the topography and the nature of the local soil deposits.

The overall setting may allow the site to be compared with other better researched areas in a similar setting, in order to estimate appropriate values for the maximum credible earthquake magnitude, and certain information about the variation of earthquake occurrence with magnitude (parameter b in Equation 1 below).

The existence of potentially active local faults is not always easy to prove; almost any site will be near fault systems, but in many cases these may not have moved for millions of years. On the other hand it is difficult to prove beyond doubt that the local faults *won't* move seismically within the period of interest. Clearly the magnitude and frequency with which a local fault can generate earthquakes has a major influence on the local hazard.

Local topography has been postulated as influencing site response due to the local reflection and refraction of the seismic waves, (see for example Chang[24]), though there are no established methods of allowing for these effects. There is also some evidence that other local features, such as the impounding of large reservoirs[25] can trigger earthquakes.

The nature of the local soil filters and modifies the earthquake motions, and this has been well quantified. It is usual to determine the ground motion that would occur in bedrock, and then to apply local modification factors due to the soil overlying bedrock at the site, as described in a later section.

Instrumental records comprise both the records of earthquake magnitudes and positions, from seismographs (teleseismic or far field records) and accelerograph recordings of ground motions (strong motion or near field records). These are often referred to as microseismic data.

The earliest seismograph records date back less than 100 years, and the early records are unreliable except for the largest events. It is only since the introduction of the World Wide Seismological Network in the 1960s (introduced to monitor underground nuclear explosions as a result of the Test Ban Treaty) that reliable worldwide data exist.

A comprehensive catalogue of earthquakes worldwide is held by the International Seismological Centre (ISC) at Newbury (of which the Ove Arup Partnership is an associate member). The catalogue is based on data supplied by seismological stations throughout the world. ISC will supply listings of earthquakes, giving their time, magnitude and position, for the area surrounding a site of interest, and these data should form the basis for establishing the magnitude/occurrence relationships described in a later section.

Accelerograph recordings of ground motions are even more limited in extent, and for many parts of the world, including North Western Europe and the Middle East, do not exist at all. The reason is of course that large earthquakes can be detected from a seismograph thousands of kilometres away, whereas accelerations sufficiently large to be detected by a strong motion instrument are produced only locally to the earthquake. Nevertheless, strong motion records are the main source of data on the attenuation of earthquake acceleration with distance and the frequency content of the motions, and so play a vital role in hazard assessment.

Eyewitness reports of earthquake damage often form an essential supplement to instrumental records. They consist of reports of the response of man-made and natural objects to an earthquake and of human perception of the motion. Such reports are often referred to as macroseismic data. The reports are usually quantified in terms of standard intensity scales, such as the Modified Mercalli scale or the MSK scale.

The interpretation of these data is an expert discipline in itself since there are a number of pitfalls, as follows:

1) Earthquake damage depends not only on the degree of shaking (the hazard) but also on the original strength of the damaged structure (the vulnerability) and the latter may be difficult to determine retrospectively.

2) Descriptions of earthquake effects may be influenced by extraneous factors, such as the novelty of earthquakes to the reporter, or his/her views about the causes of the earthquake. Also, demolition and repair carried out after the earthquake may be confused with effects of the earthquake itself. Instrumental records should be more objective.

3) It may be difficult to sort out secondhand reports (which tend to become exaggerated) from eyewitness reports.

Nevertheless, historical reports can be used to provide the following, often essential, information.

1) The extent of earthquake damage can be used to estimate earthquake magnitude and the centre of the damaged area can be used to estimate earthquake position. These can be used either to check instrumental determinations if they are uncertain or to substitute for them if they don't exist. In this way, quantitive information can be obtained about earthquakes occurring long before the advent of seismographs.

A long time series of earthquakes is important, both because of the intrinsic variability of earthquakes mentioned previously, and because the seismic hazard of a region may vary appreciably over a period longer than the time for which instrumental records are available[11].

2) The reported reduction in earthquake effects with distance can be used to estimate attenuation laws, especially in areas like the UK where no strong motion records exist[12].

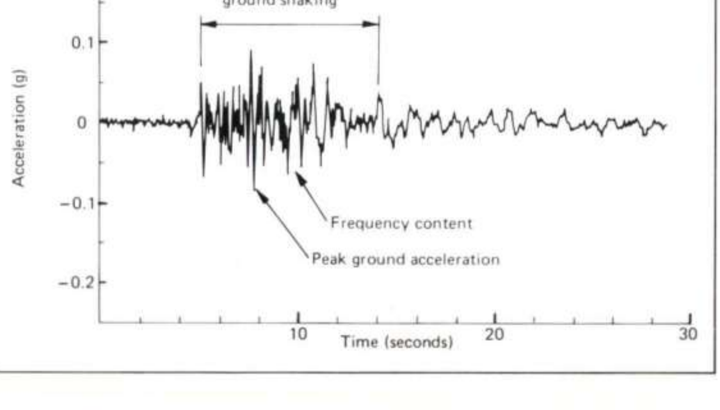


Fig.4 Typical strong-motion earthquake accelerogram (from Hart[27])

Quantification of design ground motions

A basic difficulty in quantifying seismic hazard is that the return periods of interest are at least 500 years, even for normal building structures. This was discussed at the beginning of the article. The corollary is that a design ground motion for a given site could only be deduced directly from the recordings of an accelerograph at the site if the instrument were left at the site for many hundreds of years. This can be compared with the situation for wind, where the return periods of interest are an order of magnitude shorter and design can be based on anemometer readings taken over a period of a few decades. Indirect methods must therefore be used for quantifying design ground motions.

The fact that the methods are indirect, coupled with the long return periods involved, make the determination of the motions highly uncertain. For this reason it is essential to base their assessment on the broadest possible data base, using all three data sources (geological, eyewitness, instrumental) referred to previously. Despite the impression that might be gained from the appendices to this article, sound engineering scrutiny of the data is at least as important as complex mathematical analysis.

Two methods of analysis are in common use for processing the data sources outlined in the previous section to give design figures. The methods are:

(i) Deterministic methods
(ii) Probabilistic methods.

The *deterministic methods* involve postulating a design earthquake occurring at a given distance from the site. An appropriate expression for the attenuation of acceleration with distance for a given magnitude of earthquake[13,14] can then be used to determine the design acceleration at the site.

The problem is of course in choosing the design earthquake magnitude and distance. In an area where there is a well-defined and recorded active fault system, the distance to the nearest potentially active fault may be easy to determine, and seismologists may be able to give an opinion on the largest magnitude the fault is capable of, and the frequency with which that magnitude is produced. This may be the best way of estimating the maximum credible earthquake risk to hazardous installations. In many parts of the world, however, especially away from tectonic plate boundaries, linking earthquakes to known faults is much harder, and the deterministic methods are of value primarily in checking the probabilistic methods.

The probabilistic methods involve a statistical treatment of earthquake records, in a manner first described by Cornell[15]. They assume that earthquakes occur as statistically independent events, randomly distributed in time, though they may be assumed to be clustered in space, for example along fault lines. A good description of the method is given by Cornell[16].

Two fundamental steps are needed to perform the analysis:

(1) Quantify how often earthquakes occur in the regions surrounding the site – *the magnitude/occurrence relationship.*

(2) Quantify the relationship between magnitude, ground acceleration and distance from the earthquake – *the attenuation law.*

Magnitude/occurrence relationship

The first step in deriving the magnitude/occurrence relationship is to divide the area surrounding the site into different earthquake sources – see Fig. 5. The sources can be represented either as line sources, corresponding to fault systems along which earthquakes are clustered, or areal sources, in which earthquakes are randomly distributed in space. Each source may have a different occurrence relationship.

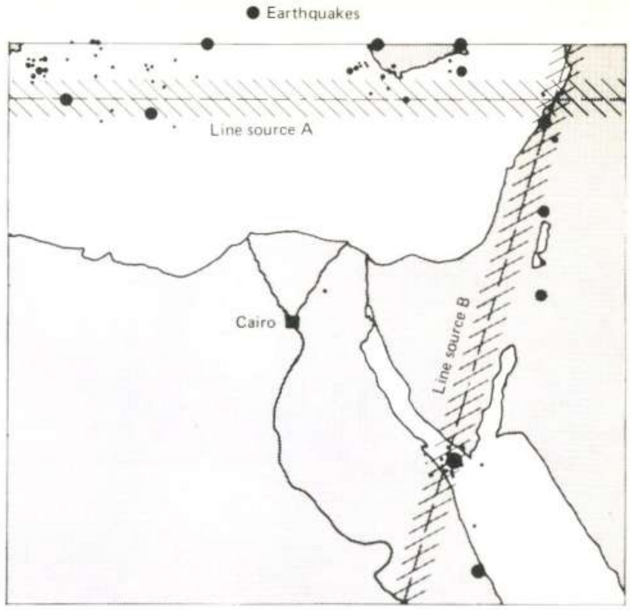


Fig.5
Line sources identified for a seismic hazard study of Cairo.

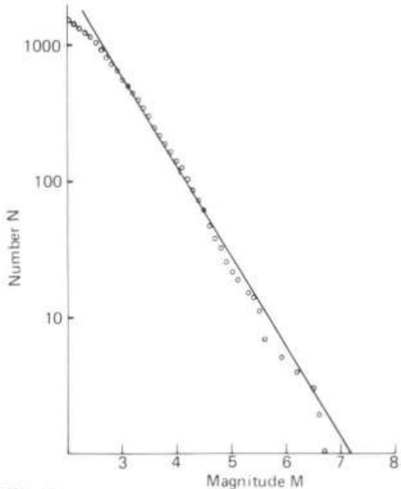


Fig.6
Magnitude occurrence relationship for Alaska (from Evernden[28])

Choosing how to divide the region into sources is done by a combination of geological arguments, on the basis of known or postulated fault systems and plate boundaries, and also by examination of the spacial distribution of recorded earthquakes. In other words, engineering judgement is involved, and it may be wise to test the sensitivity of the final result to changes in assumption about source. Cornell[16] describes this sensitivity analysis for a study of the Boston area.

The frequency of occurrence of earthquake is found[17] to be described well by Equation 1 (see Fig. 6).

$$\log N = a - bM \quad M \leq M_1 \qquad (1)$$
$$N = 0 \quad M > M_1$$

where N = number of earthquakes per year with magnitude greater than M

M_1 = maximum credible earthquake magnitude for the source

a,b are constants for a given source.

The choice of the maximum credible earthquake M_1 is usually determined by geological arguments, based on maximum recorded earthquakes in geologically similar regions. It may be the subject of considerable uncertainty, especially in areas of low seismicity. M_1 can have a major influence on the estimate of peak ground acceleration at long return periods.

a and b are determined principally by the best straight line fit of log N against M, from instrumental records of previous earthquakes. As discussed above, these data can be obtained from the ISC, supplemented by eyewitness records. Three important sources of error need to be checked:

(a) The historical record may not be complete. The further back in time the record goes, the more likely it is that smaller magnitude earthquakes will have gone unrecorded, and this is the probable reason for the falling away of data points from a straight line seen for low magnitudes in Fig. 6. Magnitudes less than four are very unlikely to damage well-built engineered structures, so the choice of a and b should be based on historically complete data for $M > 4$.

(b) There is good evidence that the rate of earthquake activity in certain parts of the world undergoes cyclical variations (see for example McGuire[11]). What has happened in the relatively recent past isn't therefore necessarily a good indication of what will happen in the future.

(c) Major earthquakes are usually linked with associated events called foreshocks and aftershocks. The analysis depends on the assumption that earthquakes are statistically independent events, so fore- and aftershocks should be excluded from the record. This involves a certain amount of judgement – when does an earthquake cease to be a delayed aftershock and become the next new event?

The simplest way to check historical completeness of record and cyclical variations is to plot the log N versus M data for a series of different historical periods. Statistically, the check is on the assumption that the earthquake occurrences are Poisson distributed with time, and a mathematical check for UK data is described by Irving[12].

Attenuation laws

An attenuation law describes the peak ground response produced by a magnitude M earthquake at a distance R from the earthquake focus. Most published laws take the form:

$$\text{Response} = b_1 e^{b_2 M} (R + c)^{-b_3} \qquad (2)$$

where b_1, b_2 and b_3 and c are empirical constants.

A great number of attenuation laws have been published[13,14]. For a region in which no direct evidence from strong motion records exists, data obtained from a geologically similar region should be used. The choice probably won't be straightforward, and the sensitivity of the final result to using different laws should be investigated. An alternative strategy is to derive attenuation laws from eyewitness reports of damage, as has been done for the UK[12].

The uncertainty in choice of law is compounded by the considerable scatter of data about the published trend lines. At the least, the analysis should be carried out for both the mean trend line and the upper bound 95% line. More sophisticated analyses[12,16] include the statistical dispersion of data about the trend line directly into the analysis. In this approach, the probability of an earthquake at a given distance causing a certain acceleration at the site comes both from a larger magnitude earthquake producing the mean acceleration, and a smaller earthquake producing an above average acceleration. In a recent study of UK seismicity,[12] the latter effect appears to have dominated the accelerations at very long return periods.

Derivation of maximum ground accelerations

Having established the magnitude/occurrence and the attenuation laws for the region affecting the site, the probability of a certain ground acceleration $\ddot{x}$ occurring at the site can be quantified as follows.

Acceleration $\ddot{x}$ can be due to either a large magnitude, low probability earthquake occurring a long way from the site, or a smaller magnitude more common earthquake occurring at close range. The rarity of the large distant earthquake is partly balanced by the smaller area in which the small earthquake must occur in order to produce an equally high acceleration.

The statistical analysis is merely a way of accounting for these effects. The earthquake magnitude needed to produce $\ddot{x}$ at a distance R from the site is given by the attenuation law. The probability of that magnitude occurring is given by the magnitude/occurrence relationship. The total probability of $\ddot{x}$ is then the sum of the probabilities of the relevant causative earthquakes occurring, summed over the entire region affecting the site.

Numerical expressions for the probability, after Cornell,[15] are given in Appendix B. A number of computer programs exist which perform the calculation, including the Arup program QUAKE, and McGuire's program EQRISK[18].

Design response spectra

In an earlier section, it was pointed out that the maximum ground acceleration is only one factor in describing the damaging power of strong ground motion. The other two factors are the frequency content of the motion and the duration of shaking.

A response spectrum describes how the peak response of a single degree of freedom system varies with the system's natural period and damping. Fig. 7 shows this schematically. The spectrum peaks where the predominant periods in the ground motion match most closely the system's natural period. A response spectrum therefore provides a good description of the frequency content of the ground motion, though the spectrum is not greatly affected by the duration of shaking.

Most earthquakes produce predominant periods of acceleration in the range 0.2 to 0.6 seconds, though some earthquakes produce predominant periods as long as 2 seconds. To put this in context, the natural periods of buildings are of the order of N/10 seconds, where N is the number of storeys, so a 5 storey building has a period of around 0.5 seconds.

It is unwise to rely on the response spectrum from a single earthquake for design purposes. For one thing, that earthquake may produce a few high spikes of response at certain frequencies, but these are unlikely to be of concern. This is because a structure resonating with the frequency of the spike would tend to shake itself out of trouble since on yielding, the natural period would change. On the other hand, a single earthquake may not contain all the periods likely to affect a particular site. Current design practice is therefore to use spectra which are the smoothed, average response of a large number of earthquakes. Codes of practice such as ATC 3.06[2] contain such spectra, which are discussed in a later section.

A large deep earthquake tends to produce longer periods than a small shallow one. Also, short periods tend to attenuate more rapidly than long periods so that the response spectrum is shifted towards the longer period range in the far field[19]. A region at some distance from a tectonic plate boundary, or other major source of earthquakes, but still within its sphere of influence, is likely to be influenced by large magnitude distant earthquakes, and so be affected by the long period shift. This shift will be important in the design of tall, and therefore long period, buildings. This is discussed in more detail in the next section.

The layers of soil overlying bedrock at the site also modify the frequency content – see Fig. 8. Deep alluvial soils amplify the longer period motions, compared with bedrock, but are unable to transmit very high accelerations, so that short period accelerations may be trimmed relative to bedrock. An estimate of the soil modification effect can be made by using a program such as SHAKE[26]. Simple methods of allowing for soil effects are provided in many seismic codes as described below, and these methods will usually be adequate for building structures.

Seismic codes of practice

At the start of this article, it was pointed out that the purpose of determining the seismic hazard at a site was to provide structures built there with an appropriate degree of resistance to earthquakes. In practice, this usually means designing the structure in accordance with a seismic code of practice. Where the site in question is not covered by such a code, it is necessary to choose a code intended for some other part of the world. If a seismic hazard analysis is used to match the site with seismic zones in the chosen code having a similar degree of seismic hazard, a similar performance (or overall risk of failure) under earthquake loading to that implied by the code should be achieved.

The seismic hazard can therefore be viewed as a relative measure, used to calibrate a site against the zoning in an accepted code, rather than as an absolute expression of risk.

Uniform Building Code (UBC) of America

About 15 building projects designed by the Ove Arup Partnership have used the seismic hazard analysis techniques described to calibrate sites in many parts of the world against the zoning of the UBC[1].

One problem with the UBC is that its seismic zoning is explicitly expressed in deterministic, rather than probabilistic, terms. Moreover, no account is taken of the long period effects of distant large magnitude earthquakes.

Nevertheless, a correlation between UBC zoning factor Z, and the 500 year return period acceleration x_{500} can be made, as shown in the table below.

500 year return effective peak horizontal acceleration at bedrock x_{500}	UBC zone	Zone factor Z
0.31g to 0.40g	4	1
0.16g to 0.30g	3	3/4
0.09g to 0.15g	2	3/8
0.05g to 0.08g	1	3/16
0.00g to 0.04g	0	1/8

The table is similar to Table 1 of Ref. 4. Its rationale is as follows. The peak acceleration given for Zone 4 corresponds to the peak acceleration for the United States given in ATC 3.06[2]. Peak accelerations for less seismic zones are factored down from this Zone 4 value in proportion to the Z factor. For example the peak acceleration for Zone 2 = 0.40g x 3/8 = 0.15g.

The choice of 500 years as the appropriate return period for building design is justified in Appendix A. It is adopted by the US Codes ATC 3.06[2] and ANSI A58.1[3].

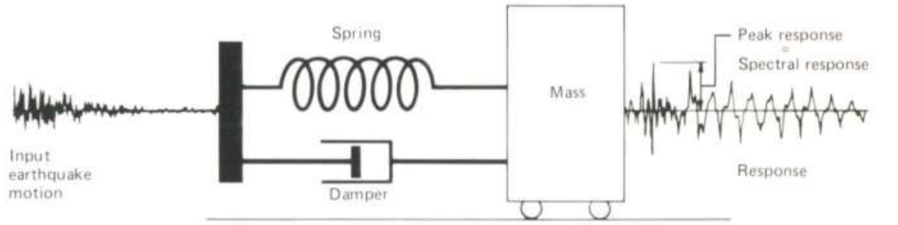


Fig.7
Calculation of spectral response

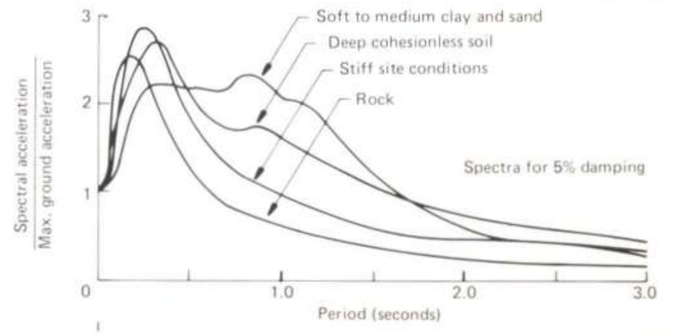


Fig.8
Average acceleration spectra for different site conditions (after Seed[14])

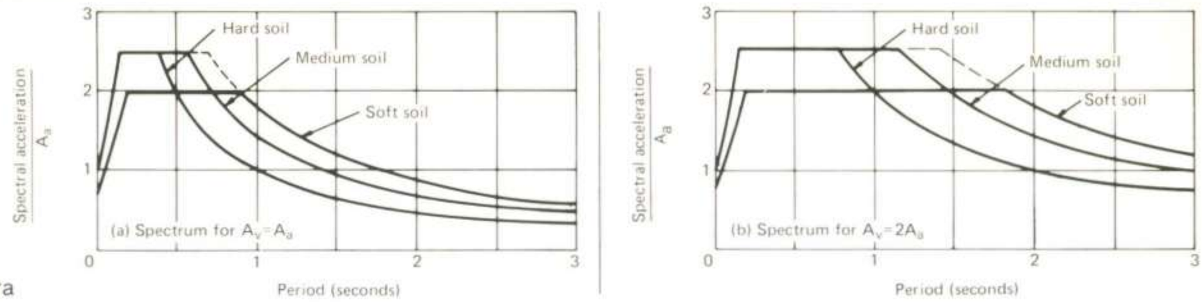


Fig.9
ATC 3.06
response spectra

Applied Technology Council Recommendations ATC 3.06

Another United States Code, ATC 3.06[2] has a zoning which is in probabilistic terms, and also explicitly allows for the effect of large magnitude, long distance earthquakes. Instead of the single zone factor Z of UBC, two factors describing seismic hazard are proposed, A_a and A_v.

A_a is equal to the 'effective' peak bedrock acceleration at a site, expressed as a fraction of g, that is associated with a 500-year return period. The 'effective' peak acceleration is defined in such a way as to discount very high frequency spikes of acceleration, which are thought to have little effect on practical building structures. The effective peak acceleration may therefore be slightly less than the true peak.

A_v is numerically equal to A_a for sites at which local earthquakes produce the predominant effects, but becomes up to three times greater for sites where distant earthquakes become important. Appendix C proposes a method for selecting A_v.

The factors A_a and A_v are used to construct the acceleration response spectra proposed by ATC 3.06. A_a determines the short period response, A_v the long period response – see Fig. 9.

The ATC 3.06 spectra also allows for the soil conditions at the site. By classifying the soil into one of three different types (broadly : hard, medium, soft) three different shapes of spectrum are obtained – see Fig. 9. Comparison with Fig. 8 shows that the ATC 3.06 spectra correspond to the field data.

Conclusions

The methods of assessing seismic hazard described in this article contain many uncertainties. However, the goal of the analysis is not primarily to quantify an absolute measure of hazard, but to ensure that structures built at a given site have an acceptable risk of failure under earthquake loading. This can be achieved as follows.

If the site in question is not covered by a satisfactory seismic code, a region should be chosen which does have a well-established, regularly updated seismic code. The seismicity of the site should then be matched against the seismicity of the code's region by performing the same type of hazard analysis for both the code region and the site. In this way, the same level of overall safety implied by the code (and it is important to check that the level is appropriate) can be achieved at the site in question. Since the level of safety is achieved by the relative, rather than absolute, level of hazard, the uncertainties and assumptions implicit in the hazard analysis become less important.

Following recent Californian practice, it is recommended that for buildings, the basis for comparison of seismicity should be the 500-year return peak bedrock acceleration. Appendix A gives some justification for the choice of 500 years as the comparison return period. It is also recommended that the effect of large magnitude distant earthquakes should be allowed for in the design of long period structures, and Appendix C proposes a method for doing so.

Acknowledgements

The contributions of David Croft to this article, especially Appendix A, are gratefully acknowledged.

References

(1) INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS. Uniform building code. ICBO, 1982.

(2) UNITED STATES. APPLIED TECHNOLOGY COUNCIL. *ATC 3.06.* Tentative provisions for the development of seismic regulations for buildings. The Council, 1978.

(3) UNITED STATES. AMERICAN NATIONAL STANDARDS INSTITUTE. *ANSI A58.1 – 1982.* Minimum design loads for buildings and other structures. The Institute, 1982.

(4) UNITED STATES. DEPARTMENT OF STATE, OFFICE OF FOREIGN BUILDINGS. Earthquake resistance design requirements. The Department, 1981.

(5) SWISS REINSURANCE COMPANY. Atlas on seismicity and volcanism. Zurich, SRC, 1978.

(6) MUNICH REINSURANCE. World map of natural hazards. Munich, Munich Reinsurance, 1978.

(7) MORTIMER-LLOYD, J.D. Earthquakes and seismic zones in the Middle East. BRE, 1983.

(8) DOWRICK, D. Introduction to seismic loadings for the Arabian peninsula. Ove Arup & Partners, 1978.

(9) DOWRICK, D. *and* REDDING, J.H. Earthquake design considerations for the Southern Dead Sea area. Ove Arup & Partners, 1978.

(10) INTERNATIONAL ASSOCIATION FOR EARTHQUAKE ENGINEERING. Earthquake resistant regulations. A world list. The Association, 1980.

(11) McGUIRE, R. Adequacy of simple probability models for calculating felt-shaking hazard, using the Chinese earthquake catalog. *Bulletin of the Seismological Society of America, 69*(3), pp.887 – 892, 1979.

(12) IRVING, J. Earthquake hazard. Central Electricity Generating Board, 1982.

(13) IDRISS, I.M. Characteristics of earthquake ground motions. ASCE Geotechnical Engineering Division Conference 'Earthquake Engineering and Soil Dynamics'. ASCE, 1978.

(14) SEED, H.B. *and* IDRISS, I.M. Ground motions and soil liquefaction during earthquakes. Earthquake Engineering Research Institute, 1982.

(15) CORNELL, C.A. Engineering seismic risk analysis. *Bulletin of the Seismological Society of America, 58*(5), pp.1583 – 1606, 1968.

(16) CORNELL, C.A. *and* MERZ, H.A. Seismic risk analysis of Boston. *ASCE Proceedings: Journal of the Structural Division, 101*(ST10), pp.2027 – 2044, 1975.

(17) RICHTER, C. Elementary seismology. W.H. Freeman & Co., 1958.

(18) McGUIRE, R. EQRISK—Evaluation of earthquake risk to site. United States Geological Society open file report no. 76 – 67, 1976.

(19) McGUIRE, R. Seismic structural response risk analysis, incorporating peak response regressions on earthquake magnitude and distance. Massachusetts Institute of Technology, 1974.

(20) CROFT, D. An alternative approach to ultimate limit state design. *The Arup Journal, 17*(2), pp.13 – 17, 1982.

(21) BENJAMIN, J.R. *and* CORNELL, C.A. Probability statistics and decision for engineers. McGraw-Hill, 1970.

(22) HOUSNER, G.W. Design spectrum. In: Earthquake engineering, edited by R.L. Wiegel, pp.93 – 106, Prentice-Hall, 1970.

(23) CROFT, D. Aseismic design in Iran. *The Arup Journal, 13*(4), pp.18 – 20, 1978.

(24) CHANG, F.K. The effects of elevation and site conditions on ground motion of the San Fernando earthquake, 1971. In: Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, 1980.

(25) INSTITUTION OF CIVIL ENGINEERS. Dams and earthquake. Thomas Telford, 1981.

(26) EARTHQUAKE ENGINEERING RESEARCH CENTRE. *Report 72 – 12.* SHAKE—a computer program for earthquake response of horizontally layered sites. University of California, 1972.

(27) HART, G. Uncertainty analysis, loads and safety in structural engineering. Prentice Hall, 1982.

(28) EVERNDEN, J.F. Study of regional seismicity and associated problems. *Bulletin of the Seismological Society of America, 60*(2), pp.393 – 446, 1970.

Appendices A, B and C follow overleaf ▷

APPENDIX A

Extreme value distribution of earthquake accelerations

Symbols

- g acceleration due to gravity
- β,N empirical constants in Weibull distribution (Equation A1)
- n exposure period, or life, of a structure, years
- T return period of a load, years
- v_n variance of an extreme annual load, for n years exposure
- x_T extreme annual load with T year return period
- y_n mean (expected) value of extreme annual load for n years exposure.

On the basis of Fig C1 – 7 of ATC 3.06[2], the distribution of annual peak accelerations follows an approximately Weibull distribution, Equation A1.

$$T = e^{\beta x_T{}^N} \tag{A1}$$

T is here the return period in years corresponding to an annual extreme acceleration x_T, and β and N are constants, depending on the region.

The best fit of the Weibull distribution to the ATC 3.06 data is shown in Fig.A1, and the corresponding constants β and N are given in the Table A1.

Table A1

Area	500 year return acceleration x_{500}	N	β
Very low seismicity (e.g. UK)	0.05g	.36	17.3
Low seismicity (e.g. UBC Zone 1)	0.10g	.37	14.2
Moderate seismicity (e.g. UBC Zone 2 – 3)	0.20g	.42	12.3
High seismicity (e.g. UBC Zone 4)	0.40g	.72	12.3

If the assumption of Weibull distribution holds, it can be shown that the extreme value distribution for an exposure period of n years is given by

$$P_n(y) = (1 - e^{-\beta y^N})^n \tag{A2}$$

$$p_n(y) = \frac{dP}{dy} = n\beta N y^{N-1} e^{-\beta y^N}(1 - e^{-\beta y^N})^{n-1} \tag{A3}$$

$P_n(y)$ gives the probability that a structure standing at a particular site for a period of n years will experience an acceleration at least as great as y. $p_n(y)$ is the corresponding probability density function*.

David Croft has shown[20] that the two important loading parameters which govern the degree of structural resistance (in our terms, vulnerability) required for a given level of safety are:

(1) the mean or expected value y_n of the peak load for the exposure period of interest

(2) the variance v_n of the peak values, which gives a measure of their statistical variation.

David Croft demonstrates[20] that for an exposure period of n years, the structural resistance R_n necessary to give a level of safety equivalent to that implied by *CP110* for dead, live and wind loads is:

$$R_n = 0.94\, y_n (1 + 4{\cdot}8\sqrt{0{\cdot}01 + 0{\cdot}77\, v_n^2}) \tag{A4}$$

The mean and variance of peak accelerations for a 50-year exposure period are shown in Table A2 using the governing data of Table A1. For comparison, typical data for UK wind loadings are also shown.

It was found that the tail of the earthquake distribution,. corresponding to very low probability events, made a significant contribution to the variance, especially for the areas of low seismicity. It was decided to ignore accelerations with return periods greater than 3000 years. This corresponds to the limit of reliability of the data shown in ATC 3.06, and a departure of 3.5 to 4.5 standard deviations from the mean. In a similar analysis of wind loadings, David Croft[20] truncated the wind distribution at the 1000 year or 3σ level; a more extreme value seems appropriate to earthquake loading, since the acceleration at that return period is only partly influenced by the maximum magnitude of earthquake assumed.

Table A2 shows that:

(1) The statistical variation is much the greatest in areas of low seismicity.

(2) The statistical variation in all areas is much greater for earthquakes than wind.

These inferences can also be drawn from Fig.A2.

*The mean and variance of a function and the other statistical terms used in this appendix are defined in standard textbooks (e.g. Benjamin & Cornell[21]).

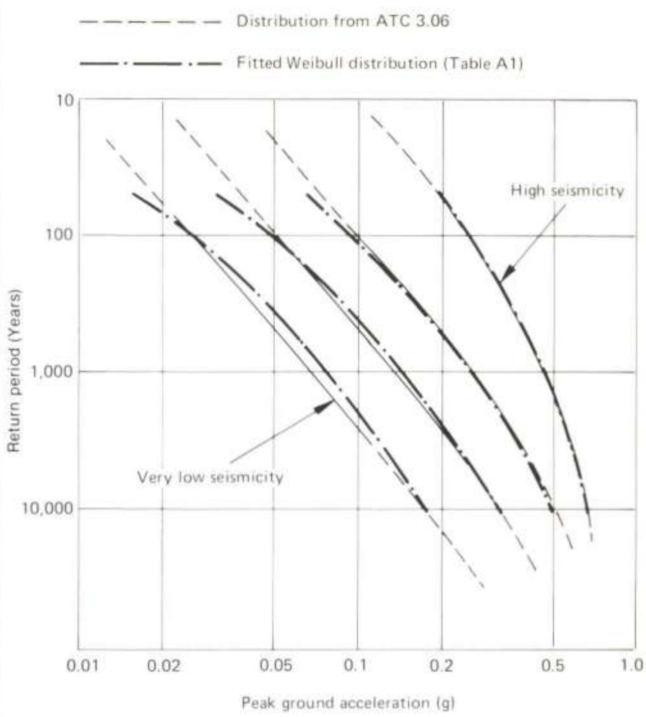


Fig.A1
Return periods for earthquake accelerations

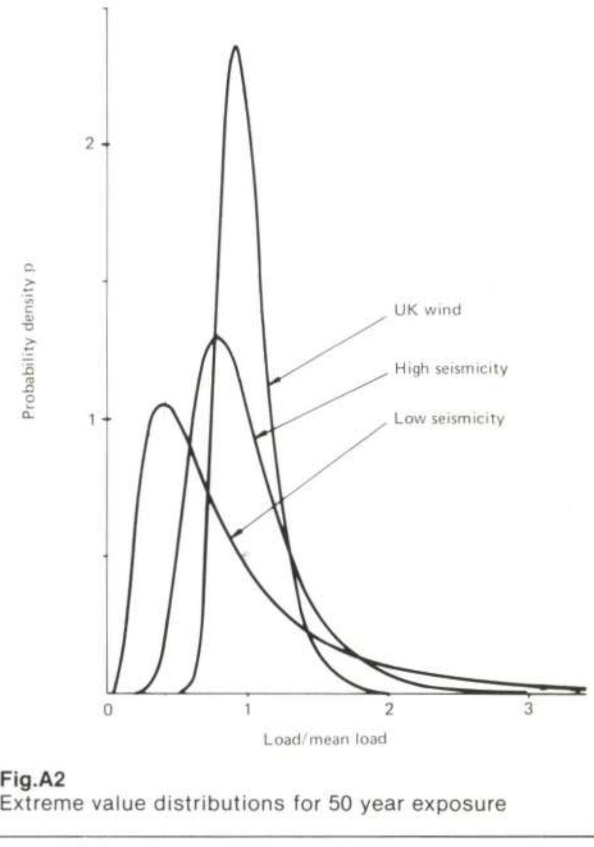


Fig.A2
Extreme value distributions for 50 year exposure

Table A2: Extreme values during a 50-year exposure period.

Area	50 years return acceleration x_{50}	Untruncated Mean y_{50}	Untruncated Variance v_{50}	Truncated to 3000 yrs Mean y_{50}	Truncated to 3000 yrs Variance v_{50}
Very low seismicity	0.016g	0.030g	0.95	0.026g	0.77
Low seismicity	0.032g	0.055g	0.92	0.049g	0.75
Moderate seismicity	0.067g	0.104g	0.79	0.095g	0.64
High seismicity	0.204g	0.253g	0.41	0.242g	0.36
UK wind loads	X	/	/	0.95X	0.20

Table A3: Required resistance for 50 years exposure period.

Area	Required resistance R_{50}	Load factors for various return periods $\frac{R_{50}}{x_{50}}$	$\frac{R_{50}}{x_{100}}$	$\frac{R_{50}}{x_{500}}$	$\frac{R_{50}}{x_{1000}}$	$\frac{R_{50}}{x_{3000}}$
Very low seismicity	0.105g	6.6	4.2	1.8	1.3	0.9
Low seismicity	0.193g	6.2	4.0	1.8	1.3	0.9
Moderate seismicity	0.334g	5.1	3.5	1.7	1.3	0.9
High seismicity	0.589g	2.9	2.3	1.5	1.3	1.1
UK wind loads	—	1.6	1.4	1.1	1.0	0.8

Table A4: Increase in load factor for an increase in exposure period from 50 to 100 years

Area	Mean load y_{100}	Variance v_{100}	Required resistance R_{100}	Increase in load factor $\frac{R_{100}}{R_{50}}$
Very low seismicity	0.035g	0.62	0.12g	1.14
Low seismicity	0.066g	0.59	0.22g	1.14
Moderate seismicity	0.125g	0.52	0.38g	1.14
High seismicity	0.285g	0.30	0.63g	1.07
UK wind loads	$1.09y_{50}$	/	/	1.06

Table A3 computes the structural resistance factor R_n of Equation A4 for an exposure period of 50 years, on the assumption that the peak acceleration is a good predictor of the required resistance. We have seen that frequency content and duration of shaking are also important factors. Assume that frequency effects are adequately allowed for in design, and neglect duration effects for the moment.

The following inferences can be drawn from Table A3.

(1) Scaling earthquake loads in relation to the 50 or 100 year return period ground acceleration would give markedly different levels of safety in areas of different seismicity.

(2) The load factor on 500 year accelerations of around 1.7 suggested by the table at first sight seems much higher than the factor of 1.0 required by ATC 3.06, even allowing for a 25% increase to account for strain-rate effects and contribution from non-structural members as quoted by Croft[20] for wind. However, it should be remembered that yielding is allowed under the action of extreme earthquake loading. Given that earthquakes cause displacement (not load) dominated cyclical effects, it seems reasonable to allow for the increase in strength between yield and ultimate. For reinforcement, a factor of (1.7/1.25) = 1.36 between ultimate and yield agrees well with the minimum factor of 1.33 required by UBC[1] for this ratio, and most reinforcing and structural steels will exceed it.

Finally, Table A4 investigates the effect of increasing the exposure period from 50 to 100 years. Areas of low to moderate seismicity require an increased load factor of 14% compared with 7% for areas of high seismicity or wind loadings. This may underestimate the required increase, due to the increased risk of prolonged shaking.

Conclusions

(1) The 500-year return acceleration provides a reasonable basis for comparison of the seismic hazard of different areas.

(2) The statistical variation of extreme earthquake accelerations experienced over a typical building lifetime is greater in areas of low seismicity than in areas of high seismicity.

(3) For all cases of earthquake loadings, the statistical variation is much greater than for wind loadings.

(4) Partial evidence has been produced to suggest that a load factor of 1.0 on 500-year return period earthquake loads provides a degree of safety broadly equivalent to that provided for in *CP110* for dead, live and wind loads.

(5) An increase in exposure time from 50 to 100 years has a greater effect on the load factors required in areas of low to moderate seismicity, than it does for areas of high seismicity, or for UK wind loading.

APPENDIX B

Quantification of earthquake hazard

The following analysis is based on Cornell[15]. It enables the return period corresponding to a given ground acceleration or velocity to be calculated from earthquake records, for a variety of earthquake source geometries.

There are three requirements for the analysis:

(1) Knowledge of magnitude/occurrence relationship (see Fig.B1) which are assumed to follow either B1 or B2.

EITHER

$$\log N = a - bM \quad \text{for} \quad M_0 \leq M \leq M_1; \qquad N = 0 \quad \text{for} \quad M > M_1 \tag{B1}$$

OR

$$\log N = a - bM + \log[1 - 10^{b(M - M_1)}] \tag{B2}$$

(2) Knowledge of attenuation law which is assumed to follow the form

$$y = b_1 e^{b_2 M}(R + c)^{-b_3} \tag{B3}$$

(3) Earthquakes are assumed to obey a Poisson disribution. See Cornell[15] for further discussion.

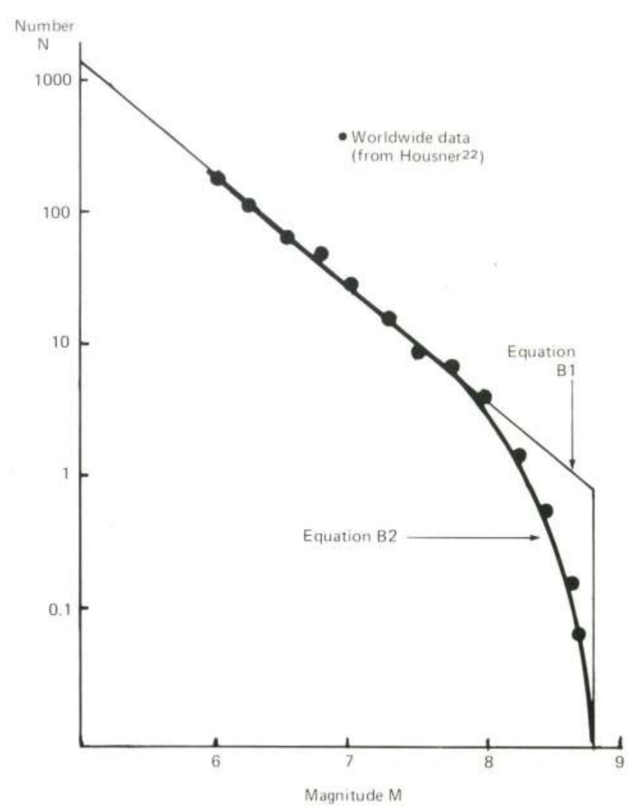


Fig.B1
Magnitude occurrence relationships

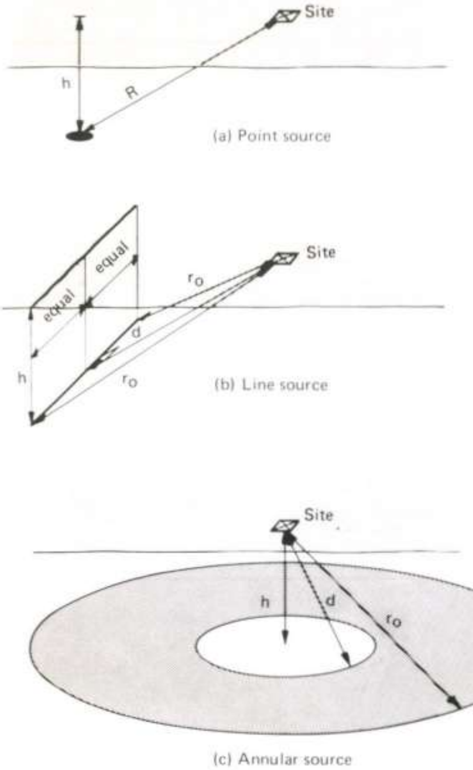


Fig.B2
Earthquake sources at depth h

Definition of Symbols

a, b	Constants in magnitude/occurrence relationship (Equations B1 and B2).
b_1, b_2, b_3	Constants in attenuation relationship (Equation B3).
c	Constant in attenuation relationship (Equation B3).
d	Distance from site to nearest point on source (Fig. B2(b) and (c))
M	Earthquake magnitude (e.g. on Richter scale)
M_o	Lowest earthquake magnitude of engineering significance
M_1	Largest credible earthquake magnitude
N	Average number of earthquakes each year with a magnitude exceeding M. N is calculated per unit length for a line source, and per unit area for an area source
R	Distance from site to earthquake focus (Fig.B2(a))
r_o	Distance from site to farthest point of source (Fig. B2 (b) and (c))
T_y	Return period corresponding to a site response y
y	Response at site (eg peak acceleration or velocity)
y_l	Lower bound response, from Equation B8
y_u	Upper bound response, from Equation B9

The following additional symbols are now defined

$$\beta = b \ln 10$$

$$\gamma = \beta(b_3/b_2) - 1$$

$$\bar{\nu} = 10^{a - bM_0}$$

for a point source $\bar{\nu} =$ average number of earthquakes per year with a magnitude of at least M_o

for a line source $\bar{\nu} =$ average annual number per unit length

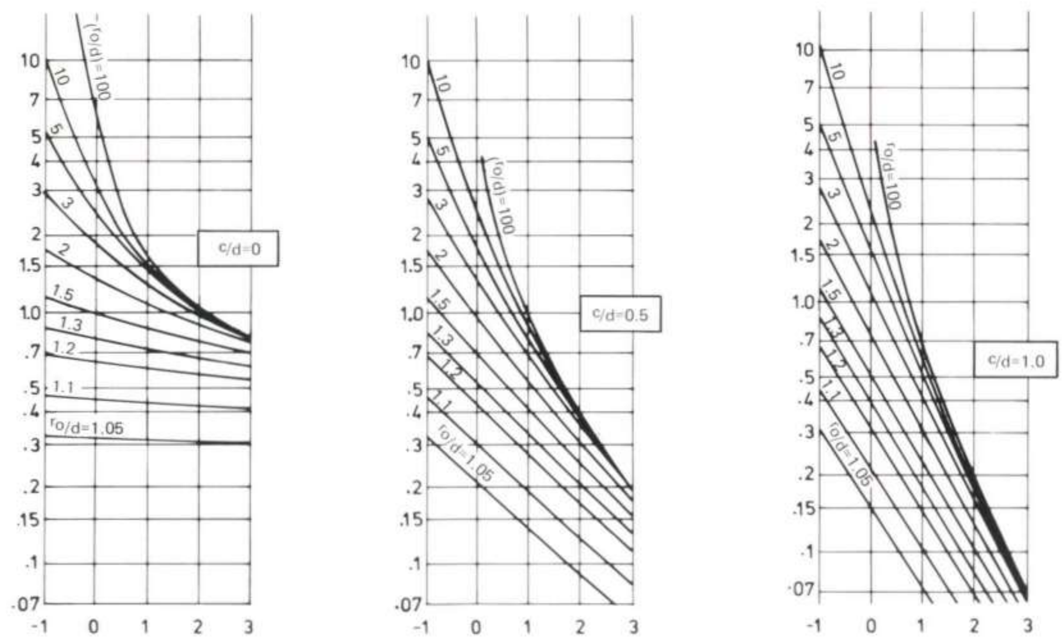


Fig.B3
Numerical values of Q in equation B5

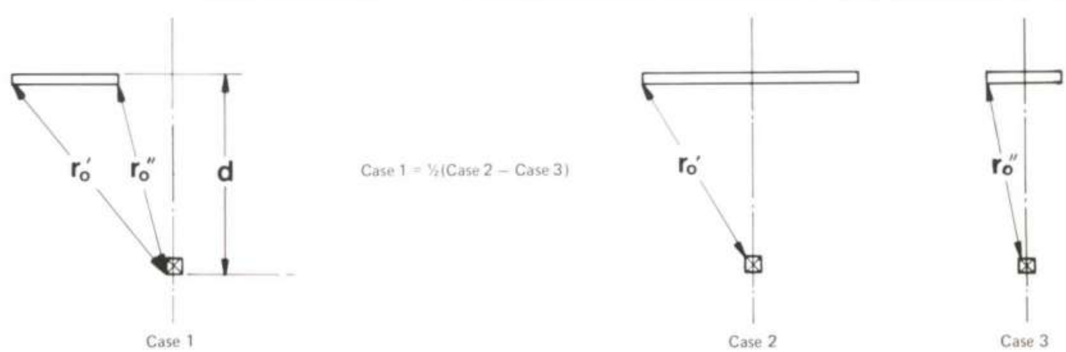


Fig.B4
Asymmetrical line sources

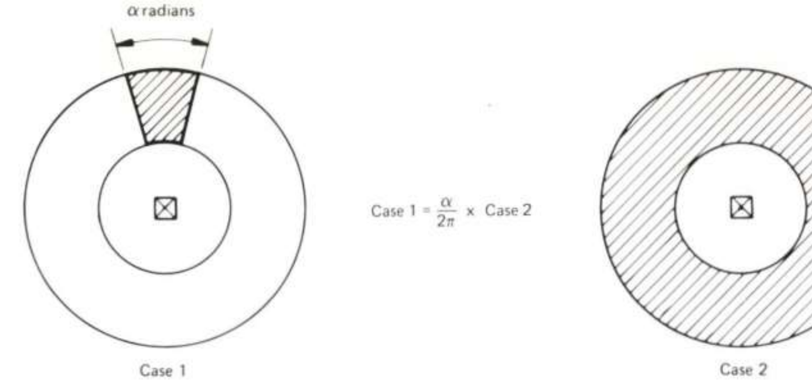


Fig.B5
Annular sector areal source

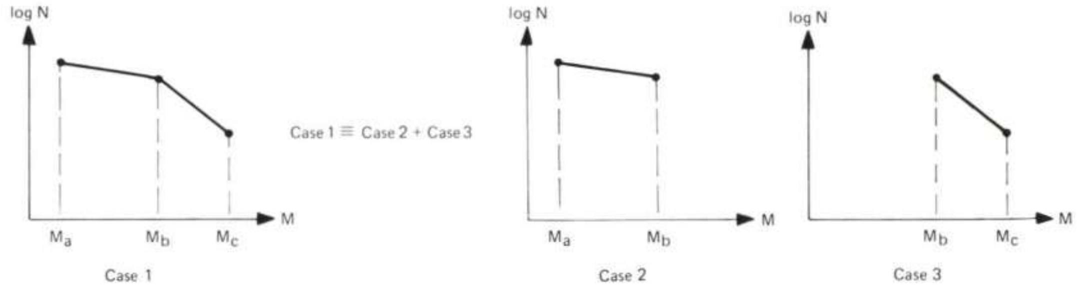


Fig.B6
Bi-linear magnitude occurrence relationship

for an area source $\bar{\nu}$ = average annual number per unit area

$$C_1 = b_1^{\beta/b2} e^{\beta M_0}$$

$$C_2 = 0 \quad \text{if equation B1 holds}$$

$$C_2 = e^{-\beta(M_1 - M_0)} \quad \text{if equation B2 holds}$$

G_1 G_2 are geometry factors depending on the shape of the source, and are defined below.

Geometry factors G_1 and G_2
G_1 and G_2 are now given for three basic geometries of source, though Figs. B6 to B8 show how other geometries can be derived from the basic ones.

Point source (Fig. B2(a))

$$G_1 = (R + c)^{-(1+\gamma)} \qquad G_2 = 1 \tag{B4}$$

Line source (Fig. B2(b))

$$G_1 = (2/d^{\gamma})Q[\gamma, r_0/d, c/d] \quad G_2 = 2d\sqrt{(r_0/d)^2 - 1} \tag{B5}$$

where $Q[\gamma, r_0/d, c/d] = \displaystyle\int_0^{\sec^{-1} r_0/d} \frac{\sec^2\theta \, d\theta}{(\sec\theta + c/d)^{\gamma+1}}$, which can either be evaluated by numerical integration, or can be read off Fig. B3.

Annular source (Fig. B2(c))

$$G_1 = \frac{2\pi}{(\gamma-1)(d+c)^{\gamma-1}}\left(1 - \left(\frac{d+c}{r_0+c}\right)^{\gamma-1}\right) - \frac{2\pi c}{\gamma(d+c)^{\gamma}}\left(1 - \left(\frac{d+c}{r_0+c}\right)^{\gamma}\right) \tag{B6}$$

$$G_2 = \pi d^2((r_0/d)^2 - 1)$$

Quantifying Return Periods
It can be shown that the following equation holds

$$1/T_y = \bar{\nu}(C_1 G_1 y^{-\beta/b2} - C_2 G_2) \tag{B7}$$

This approximation is sufficiently accurate for $T_y > 20$ years, and $(M_1 - M_0) > 2$ provided that

(i) $$y > y_l = b_1 e^{b2 M_0}(d + c)^{-b3} \tag{B8}$$

This condition says that Equation B7 only holds for accelerations greater than that produced by the lower bound earthquake acting at the nearest point on the source from the site. To deal with lower accelerations, M_0 must be lowered, which alters $\bar{\nu}$, C_1 & C_2.

(ii) $$y < y_u = b_1 e^{b2 M_1}(r_0 + c)^{-b3} \tag{B9}$$

This condition says that Equation B7 does not hold for accelerations higher than that produced by the maximum credible earthquake acting at the farthest point on the source from the site.

To deal with larger accelerations, r_0 must be lowered; ie. that part of the source which can't produce at least y_u must be neglected. For a point source, of course, y_u is the absolute maximum acceleration, and there is no risk of its being exceeded.

Combination of sources of different types
The overall contribution to earthquake hazard from a number of different sources can be added together to produce the overall hazard. Equation B7 then becomes

for n sources $$1/T_y = \sum_{i=1}^{n} \bar{\nu}_i (C_{1_i} G_{1_i} y^{-\beta_i/b2_i} - C_{2_i} G_{2_i}) \tag{B10}$$

By this means, sites affected by a combination of point, line and annular sources can be analyzed. Also, different source geometries (Figs. B4 – B5) or different magnitude/occurrence laws (Fig. B6) can be modelled.

APPENDIX C

Determination of factor A_v from ATC 3.06[2]
As discussed in the main part of the article, the procedures proposed by ATC 3.06 include a factor A_v to allow for the long period effects of distant, large magnitude earthquakes. ATC 3.06 gives values of A_v for sites in the USA; the following approximate procedure is recommended for sites elsewhere. The procedure is based on information given in the commentary to ATC 3.06.

(1) In regions where local earthquakes have the predominant effect on peak ground accelerations, take $A_a = A_v$. Such sites can be defined as those where the removal of a distant source of earthquakes has little effect on the computed value of A_a.

(2) Where a source more than 100 km from the site has a significant effect on the seismic hazard of the site, the peak ground acceleration A_a^* should be calculated at the centre of this distant source.

(3) If the source is near a tectonic plate boundary, and the region is thought to be similar to California, A_v at the site should be taken as

$$A_v = \frac{A_a^*}{2^{D/130}} \tag{C1}$$

where D is the distance in kilometres from the site to the nearest point on the distant source.

(4) If the source is in an intraplate region similar to the mid-west and eastern states of the USA, take

$$A_v = \frac{A_a^*}{2^{D/130}} \quad \text{for } D < 130 \tag{C2}$$

or

$$A_v = \frac{A_a^*}{2\left(\frac{D+130}{260}\right)} \quad \text{for } D > 130 \tag{C3}$$

(5) A_v should never be taken as less than A_a at the site.

Strengthening of existing filler joist floors

Norman Beaton

This paper was given at THE ARUP PARTNERSHIPS seminar 'Innovation in practice', November 1983.

The problem

MEPC Ltd. developed the site now occupied by the BPCL headquarters in Buckingham Palace Road. As part of the planning permission to develop they had to redevelop the site occupied by a building called Chantrey House. This project set Arups an unusual problem in 1975. The building facade was 'listed' and the client needed to develop the site commercially. This involved opening up a load-bearing wall building and providing floors that could support an enhanced imposed loading for offices.

The building was constructed around 1910 as flats, but had subsequently been used as offices by several small firms. The existing structure of the building was basically steel filler joist floors with breeze concrete infill supported on load-bearing brick walls.

Our initial investigations of the existing floors suggested that the allowable imposed loading that the floors could sustain was in the region of 1.5 to 3.5kN/m^2 which was considerably less than the 5.0kN/m^2 required by the client.

When the existing timber floor covering was removed, the top flange of the filler joist was found level with the top of the breeze concrete, presenting a fire problem.

Alternative solutions

Initial consideration was given to gutting the building, and leaving the external walls in place. This was thought, however, to be an expensive and slow construction.

The alternative of replacing the internal load-bearing walls with a steel frame and somehow strengthening the existing floors seemed a more attractive proposition. The load-carrying capacity of the filler joists could not be increased by introducing intermediate supports because of the lack of head-room, the need to air-condition the building, and the general circulation requirements.

The proposal

If the existing timber floor was replaced by concrete, we believed we could justify the existing filler joists by using them in composite action with this concrete, first having reduced the stress in the filler joists by propping them at mid-span. This would also provide the required fire resistance.

Our approach

We asked around in Arups to see if this sort of thing had been done before, but much to our surprise it had not. We therefore embarked upon procedures to establish the viability of the proposal.

The first thing we did was to talk to the District Surveyor and get his initial reaction. In principle the proposal was accepted but he felt that an allowable stress in the steelwork for bending of 100N/mm^2 was more appropriate than the 125N/mm^2 we had assumed. He was willing, however, to be convinced later on this point.

We agreed that the only way to be certain was to carry out a testing programme on the steel to determine its strength and weldability. The level of testing was set at sampling 2½% of the number of filler joists, including random in situ hardness testing.

The result of this testing was that the steel was found to be of a weldable and consistent quality. The yield stress of the samples varied from 230N/mm^2 to 324N/mm^2 which compared favourably with the minimum guaranteed yield stress of 255N/mm^2 for present day Grade 43 steel. It was agreed with the District Surveyor that we would use present day stresses reduced by 10%. This gave an allowable stress in the steelwork for bending of 149N/mm^2 which was greater than we had first assumed, and confirmed the viability of the scheme.

A detailed survey was then carried out of the various sizes of filler joist related to span, so that we could determine by calculation the amount of jacking to the existing filler joist that was required.

The loads anticipated in the jacking system were small, but we were uncertain how much additional load-carrying capacity would be needed to overcome the locking effect of the breeze concrete. There was a large amount of shoring and jacking required, and as we were looking for the most simple and economic solution. *Acrow* props were the sort of thing we had in mind but due to their limited load capacity on storey height lifts and the unknown load that may be imposed in the props, we decided to carry out a test using hydraulic jacks. The test proved that the locking effect would increase the load in the prop by 60% over and above that calculated. The centres of the props were adjusted accordingly.

The construction

The sequence of construction adopted for one bay of slab was:

(a) Weld shear studs to the top flange of the filler joists.

(b) Take precise levels of the top of the filler joists.

(c) Jack up the floor slab the required amount.

(d) Lay mesh on top of the existing slab.

(e) Cast a 75mm thick concrete topping.

(f) Deprop after seven days or when the concrete reached a strength of 13N/mm^2.

At first the contractor was nervous about carrying out the work, but after experience did it successfully and with confidence. The time taken to destress a bay of slabs was between 1 and 2 hours.

Problems that arose and points to note

The main problem was defining the boundaries for propping due to the effect of the locking action of the breeze concrete and the need to maintain support to the external walls. This was overcome by cutting out breeze concrete when the joist ran parallel to external walls and subsequently replacing it with reinforced concrete after the strengthening operation had taken place.

Great care was needed to ensure that the props were placed vertically, were in good condition, and were not bent. All these points would have reduced their load-carrying capabilities.

Conclusions

The building has been completed for some three years and has operated satisfactorily with no signs of distress. Given the same situation again, this form of strengthening of existing filler joist floors should be seriously considered.

Fig. 1
General view of Chantrey House (Photo: Norman Beaton)

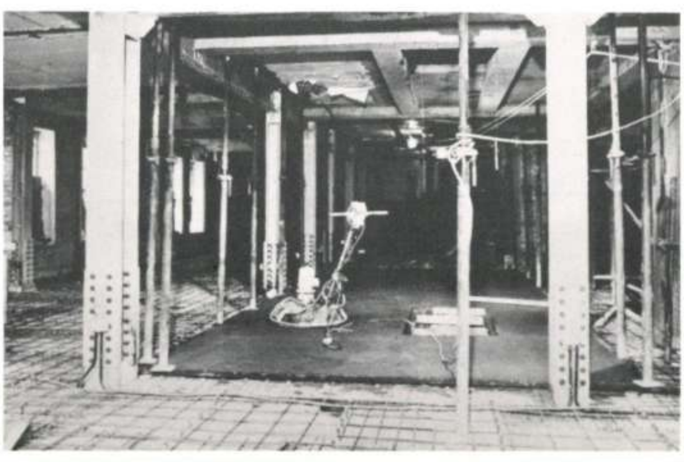
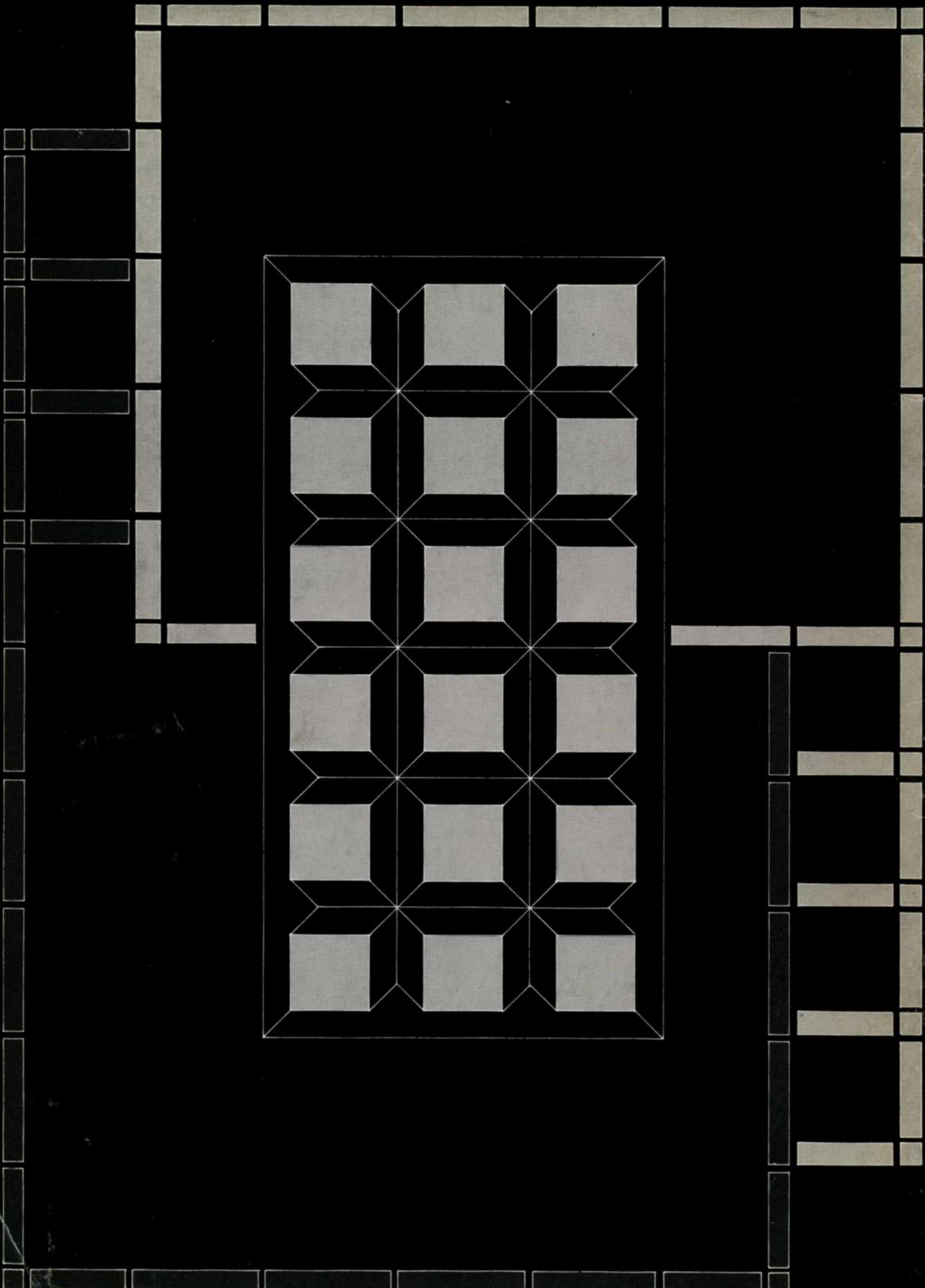


Fig. 2
Strengthening of existing floor (Photo: Courtesy of Bovis Ltd.)



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