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**Contents**



Front cover: Angel Court (Photo: Frank Gadd)
Back cover: Buxton Opera House (Photo: Arup Associates)

# The Salhia complex, Kuwait

**Architects: Kuwaiti Engineering Group**

Malcolm Threlfall
Donald Ryder
Colin Wood

'Trees grow in Kuwait'. So proclaims the brochure of the Salhia Real Estate Company, owners of this complex. With no less truth but rather less sales licence the brochure goes on to say. 'In Kuwait everything is possible'. The aim of the owners is to create a distinctly 'up market' commercial centre and hotel. The main aim of this article is to show that, in terms of building structure, many of the up-to-date techniques of Western industry are being used successfully in the Middle East. Much has been written and heard of poor quality construction in that part of the world, and it certainly exists. However it is equally true that, with proper attention, work of the best quality can readily be achieved.

### Two-part complex

As can be seen from the artist's impression (Fig. 3) the complex has two distinct parts: the longer, lower block of the Commercial Centre linked to the more compact and taller Meridien Hotel. As shown in the section (Fig. 1) the accommodation in the Commercial Centre is basically quite simple: five storeys of office over three of shops over two basements of car parking. The hotel accommodation (Fig. 2) is naturally more varied and interesting. The 13 floors of bedrooms (336 rooms and 35 suites) are topped by a super night club and swimming pool. Underneath are restaurants, ballroom (conferences), meeting and service rooms. The tight road frontage and site area of the hotel mean that the main taxi and car access is through the car parking area of the Commercial Centre. The high standard of finishes which are now being applied to both buildings are exemplified by the swimming pool. The interior designer's drawings of the intriguingly shaped pool make it look as though it awaits Cleopatra.

By Kuwaiti law the legally responsible designers (the 'engineer of record') must be a registered Kuwaiti engineering office (which naturally means that it has a majority Kuwaiti ownership). As elsewhere in the Middle East the 'engineer' is the principal professional. Architects work in engineers' offices – even where the owners are architects! For this complex the engineer of record is the Kuwaiti Engineering Group (KEG) with whom Arups are 'associated for structural design'. KEG are multi-professional and have carried out the architectural and services engineering design and supervision. Arups' involvement in the project was the result of another Kuwaiti law that makes the contractor responsible for design as well as construction, for a period of 10 years. The conceptual structural design of the Commercial Centre was produced by a Scandinavian consultant who, together with a European architect, was at that time 'associated' with KEG. After he had started work on site in February 1976, the contractor (Ahmadiah Contracting and Trading Co.) asked Arups to check the structural design. This led to Arups being asked to take over the detailed design and construction drawing stages and to provide a resident structural engineer. For the hotel which started on site some 18 months later, Arups are providing a full structural service from inception to completion.

Both buildings are founded on structural rafts. Except for localized soft areas the subsoil of Kuwait is a very compact, slightly cemented sand. As can be seen in Figs. 4 and 5 the ground stands well in open excavation. In Kuwait the Government Research Station provides a free site investigation service. The operators must get very bored by their endless logging of 'N' values in the 50–100 range. Despite these high readings the Municipality had a limiting bearing pressure of 3 kg/cm². For this site they were persuaded to agree to 5 kg/cm². The more interesting raft is the one for the Commercial Centre where the 18 x 8.4 m grid imposes rather widespread column loading (maximum 12,200 kN) and core positions. The perimeter loading, which is spread longitudinally by the retaining walls, also has to be spread laterally by the raft. On the north side the external line of construction is also the building line so there is a minimal toe projection. The effect of this on the raft form can be seen in Fig. 1 by comparing it with the south side where a better toe could be provided. A similar 'edge' problem occurs down the centre of the building where the large services duct creates a very thin section of raft just where pressures are highest and easy load distribution most desirable. Distribution under the duct had to be sacrificed and a 100 mm polystyrene compressible layer provided underneath the duct to prevent the imposition of unbearably high pressures on the thin slab. The alternative of 'bending' the thick raft under the duct was unattractive for both detailing and constructional reasons. The water table lies 1.5 m above the lower basement level. Wellpoint dewatering can be seen in Fig. 4.

Whilst the more concentrated loading in the hotel made for a fairly straightforward raft the hotel site provided some interesting

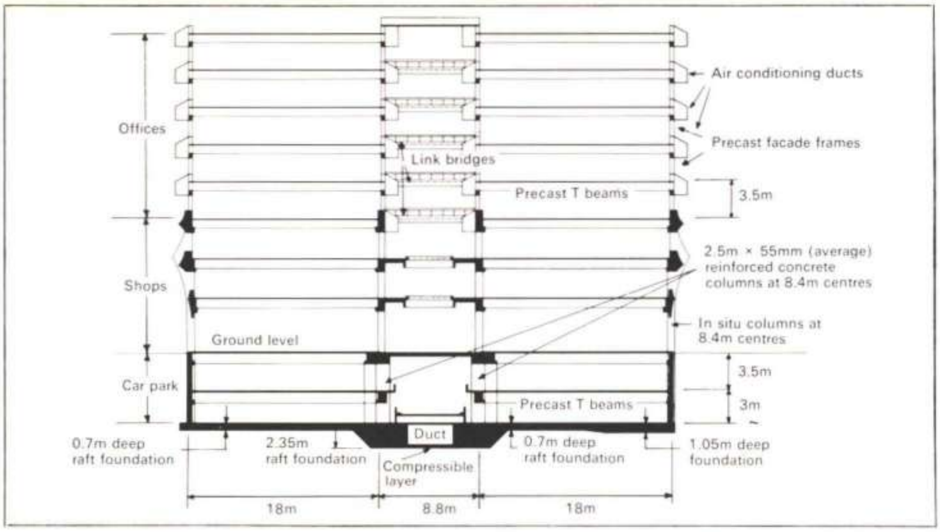
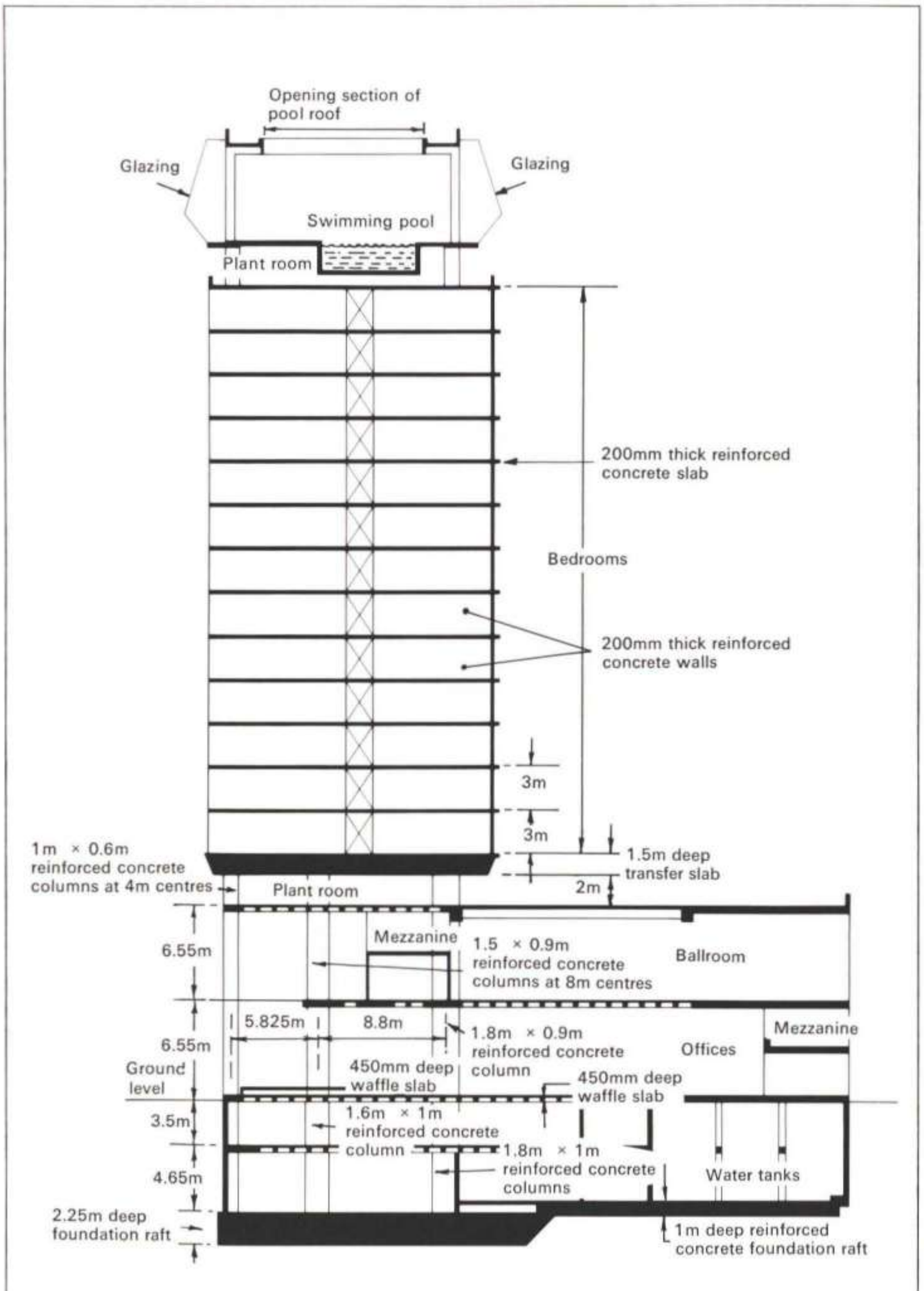



**Fig. 1**
The Commercial Centre: section

**Fig. 2 below**
The Meridien Hotel: section




upholding problems. In Fig. 5 the close boarded sheeting with soil anchored soldier piles was necessary because of the busy nearby roads (Fig. 6), major underground services in the sidewalk, supporting tower cranes and creating working space. On the south side of the site lies an extensive graveyard (Figs. 6, 7 and 8). Positive upholding was not considered essential along here but a local quirk in the soil formation led to occasional visitations from the resident freeholders. It will soon be 25 years since the last interment and the ground will then be covered as a landscaped area.

Another interesting upholding problem can be seen in Fig. 5 where the basement of a seven-storey building is shown exposed. The hotel is built tight up to this building and the new foundation level is 4.4 m below the existing one. The block of soil below the existing building clearly had to be confined whilst excavation and new construction took place. The principles of the method used are clear from Fig. 5. The building was left on a shelf of soil whilst the centre strip of the raft was constructed. At the same time holes were drilled tight up to the building and steel soldier piles concreted in (note the tops protruding). Temporary concrete pedestals were built on the top of the central raft strip and two levels of raking steel props strutted between the pedestals and the soldier piles as the soil shelf was dug out. The soldier piles were concreted into the permanent construction and the temporary props removed as the permanent floors and lift cores rose to take over their function.

**Site layout**

The general site layout for construction purposes can be appreciated from Fig. 7. The south (graveyard) side was always inaccessible and, after a short period, the opposite long side on the north was barred. The long site had thus to be serviced from the two short ends, some 250 m apart. The centre strip of the ground floor of the Commercial Centre was designed to be built quickly to act as a construction bridge along which tower cranes and vehicles could run. In the permanent buildings this deck forms the shopping mall and also has to carry fire engines. Construction of the hotel could not be readily started until enough of the Commercial Centre had been built to allow major access to be cut off at the hotel end. The Commercial Centre was started at the hotel end but progress was not quite fast enough. A temporary bridge was built across the hotel site and the hotel built around it up to ground floor level. Despite the hurried importation and stacking of building materials and the posting of a tower crane on the mall roof (suitably propped) in the Commercial Centre, hotel construction was delayed by a few weeks.

The superstructure of the Commercial Centre can be described as bold and simple but with interesting connotations. Fig. 10 shows that the 200 m total length is subdivided into five similar sections. The centre section on the north side embodies the grand entrance complete with double spiral staircase, water feature, GRC panelling and long span in situ flooring. Elsewhere, most floors are built from 18 m span precast, prestressed 'T' section floor units with a 100 mm structural topping. The units were made locally in a new factory started up for this scheme. The sub-contractor was Al-Rabiah International (Arinco) in association with Hurks-Beton of The Netherlands who designed both these units and the precast column frames which support the

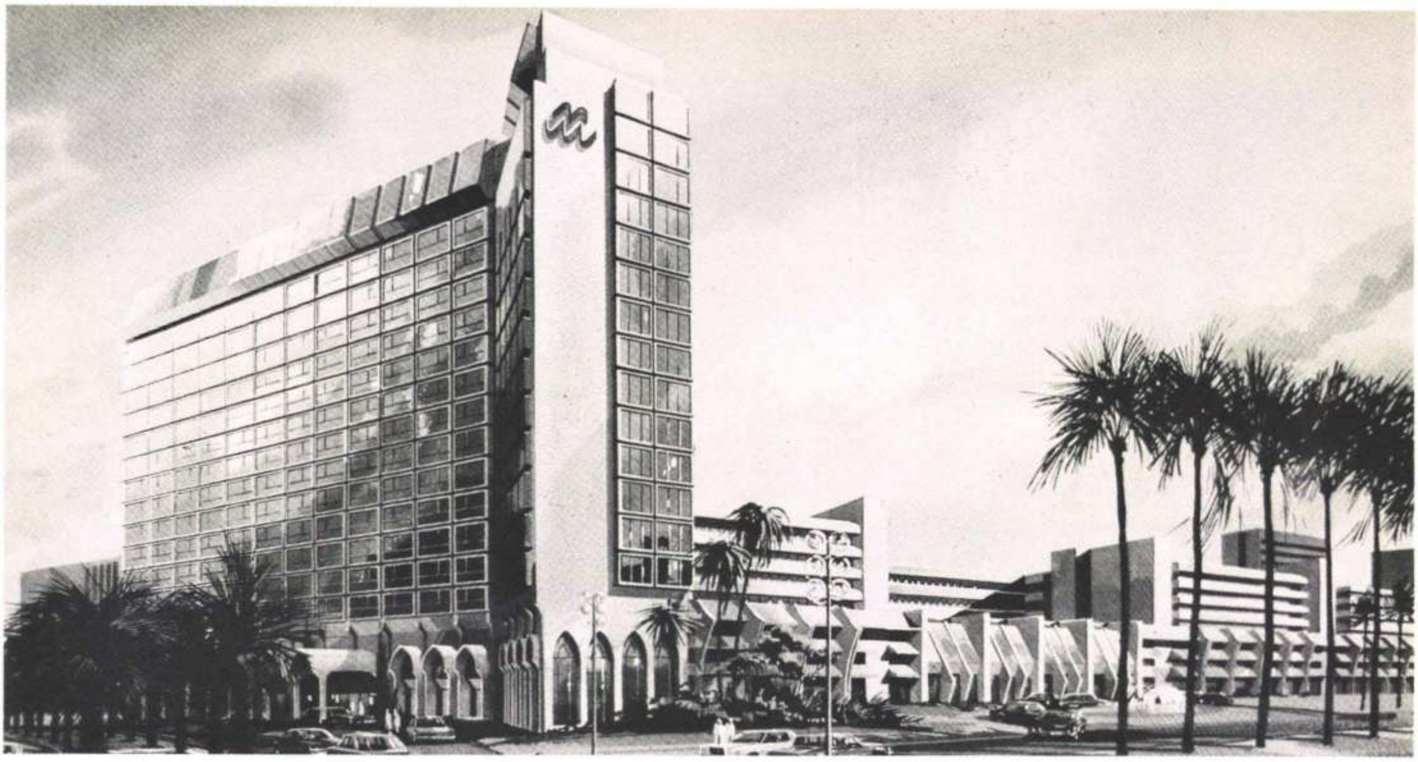


**Fig. 3**
Artist's impression of the Salhia Complex (Illustration reproduced by courtesy of the Salhia Real Estate Company)

**Fig. 4**
The excavation at 26 December 1976, showing wellpoint dewatering (*Photo:* Oscar Matri, Kuwait)

**Fig. 5**
The excavation at 23 November 1977 (*Photo:* Oscar Matri, Kuwait)

floors over the five office floors. This can be seen in Fig. 10 which also shows the external longitudinal air conditioning distribution ducts concealed behind aluminium covers. The prestressing was by pretensioned strand and the concrete was steam-cured. Lifting was at one day and a concrete strength of 250 kg/cm². Several units were satisfactorily test loaded. Differential camber was up to 50 mm but adequately explained by differential times/strengths at lifting and transporting, and adequately countered by judicious positioning and the thick topping.

The lift/stair cores which stabilize the structure are shown in Figs 7 and 8. As can be seen they were slid ahead of the main construction. Losinger of Switzerland were the sub-contractors for the sliding formwork and expertise. The conceptual design had positioned the expansion joints of the building just inside one of the outside walls of the core. In the free-standing mode the cores thus had one loose wall, so temporary steel braces were fitted to stabilize the box. In addition to this vertical severance the cores had the considerable vertical discontinuity illustrated in Fig. 8. The large gaps in the lower walls provide the necessary horizontal circulation at car parking and main shopping floors. So much of the basement storeys is removed that sliding was started at ground floor level. The photograph shows the heavy steel propping which was required temporarily until the core wall was completed and supported by the complex column on the outside wall. Other temporary devices such as false concrete columns, later cut off, were required to give structural integrity to the core whilst it was at low level and before the solid upper parts had been cast.

Concreting of the cores had to stop during the five hot summer months from May to September. One core had to be abandoned when the heat caused the concrete to dry out so quickly that the moving shutter dragged out large cavities, cracks and surface imperfections. After the summer and the completion of the other cores the top of the abandoned core was cut down and the slide restarted. Despite the ecumenical sacrifice of a sheep and erection of a cross on the shutter, this core remained a jinx and climbed only slowly and haltingly to its summit.

**Fig. 6**
The hotel under construction on 2 January 1979, with the Commercial Centre behind (*Photo:* Oscar Matri, Kuwait)

**Fig. 7**
The general site layout, with six slipformed cores cast and six still to be cast (*Photo:* Oscar Matri, Kuwait)

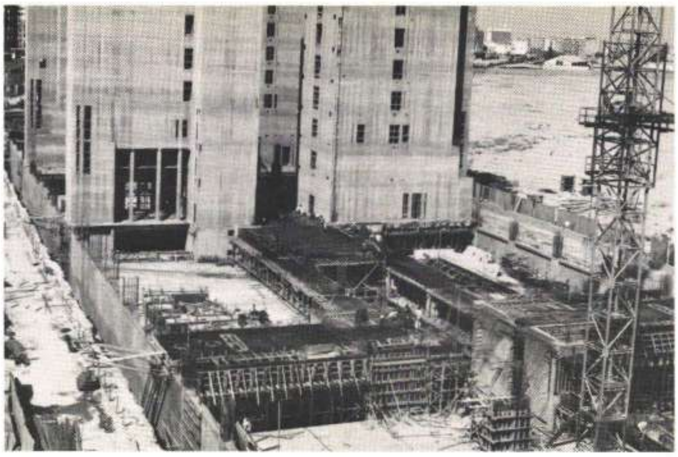


**Fig. 8**
Close-up of slipformed cores under construction, 17 April 1977 (*Photo:* Oscar Matri, Kuwait)

The design concept for the Commercial Centre called for 28 day concrete cube strengths of 350 kg/cm² but this was reduced to 300 kg/cm² and regularly achieved. All in situ concrete was supplied ready mixed with water added at site. The supplier was the National Industries Company, a Government organization which also manufactures bricks, precast concrete and many other products. For the thin wall sections the design requirement was limited to 200 kg/cm² to allow for the expected hydration losses from the large exposed vertical surfaces which are not easy to cure properly. Curing of vertical elements was by watered hessian; horizontal surfaces were given a sprayed curing mèmbrane.

Whilst Arab clients (excluding this particular one) do not always appreciate the difficulties, manpower requirements and cost of the design process, they certainly appreciate the need for good site supervision. The cost of the supervision is often greater than the cost of design. On both these sites there were large supervisory teams. Arups' resident engineers were responsible for the structure but worked as part of teams led by KEG architects. The only other item of particular structural note in the Commercial Centre is the complex shape of the external column/beam framework between ground and third floor level. These were cast into fibre glass moulds manufactured by the British firm of Barnes Reinforced Plastics, who claim that the column moulds are the biggest ever made in fibreglass and who also made the moulds for the waffle slab floors of the hotel. The load on each third floor beam span of 8.4 m is 900 tonnes. The structure was designed for seven storeys of offices but only five have been built.

**Some statistics**

| Item | Centre | Hotel |
|---|---|---|
| Approximate floor area (m²) | 80,000 | 32,000 |
| Number of floors | 10 | 19 |
| Vol. of concrete (m³) | 37,000 | 21,000 |
| Weight of reinforcement (tonnes) | 3,800 | 2,850 |
| Approximate cost (£) (excluding furniture and decoration) | 24m. | 17m. |
| Construction period (months) | 39 | 30 |

Fig. 2 shows the three-layered structural form of the hotel. At the roof the frame sits on the cross walls of the bedrooms at 4 m centres. Below these rooms the vertical structure has to be opened up to a maximum internal grid of 8 x 8.8 m to provide the necessary space for the public rooms. The resultant horizontal load shifting at first bedroom level is achieved through a thick transfer slab. The equivalent gathering and horizontal shifting of the vertical drainage and other services descending from each pair of bedrooms is similarly effected through the plant room/services level immediately below the structural transfer slab.

Basement walls are propped via the floor slabs in the usual way. Normal reinforced concrete and water bar construction with external damp proofing paint have proved adequate to resist ground water but slight leakage from the extra pressure in the water tanks will require these to be lined with either fibreglass or a loose butyl sheeting (in the Commercial Centre a few of the construction joints have wept and are being cut back and sealed). The lower suspended floors are mainly constructed as 450 mm thick waffle slabs which are simply painted and left exposed in the basement service areas. An excellent quality of finish was obtained. Whilst the waffles were formed from imported plastic moulds the propping system was traditional Kuwaiti close centred timber framing – it is surprising that more sophisticated proprietary systems are not more widely used, as no timber is produced locally. Because of the cranked plan form and irregular support system, several areas of slab were solid and many had six layer patterns of reinforcement. Both reinforcement fixing and formwork construction were sublet to specialist contractors.

Because of the heavy dead weight of the transfer slab and the extended propping distance, the contractor requested that the slab be designed for casting in two layers. The first of 500 mm thickness was poured onto normal falsework and allowed to strengthen so that it could be used to fully support the second pour of 1 m thickness. Whilst this method obviously increased the reinforcement content (to a ratio of 4 m³ per tonne, compared with 9 in the foundations, 8 in the rest of the Hotel and 10 in the Commercial Centre) there were obvious savings in falsework, time and convenience. As the main contracts are cost plus, a detailed cost benefit analysis may not have been made.

Cross wall construction is strangely somewhat unusual in Kuwait where the local mode is a 4 x 4 m or 6 x 4 m gridded frame with infill blockwork. Whilst tunnel formwork had been very successfully used on one previous, low rise project, the Contractor (who was appointed before the design process and thereby beneficially available to agree methods

**Fig. 9**
South-west side of the hotel under construction, showing arrangement of bedroom windows, on 30 November 1978 (*Photo:* Oscar Matri, Kuwait)

of construction in advance) was initially a little dubious of its application on this high rise and high level scheme.

However, after a little consideration, he became enthusiastic and adopted the French Outinord system of formwork which folds and leaves the slab propped at mid-span. This enabled stripping to be allowed at 36 hours (for an early morning start on the second day after casting) or 75 kg/cm² strength (which was always amply exceeded). This particular shutter system was useful for dealing with the cranked bedroom walls on the south west side which are angled to direct the view parallel with the end of the Commercial Centre Fig. 9). The ballroom roof provided a useful resting and repair platform for the steel shutter units. Slabs are 200 mm thick to deal with various odd bays of larger span and also to allow the tops of the bathroom floors to be sunk to receive drain pipes, whilst keeping the flush soffit necessary for the tunnel formwork. An excellent standard of finish ready for direct decoration has been produced. Blockwork and services have followed up the structure very rapidly. Electric conduit, socket outlet boxes, etc., were cast into the concrete in the usual manner of system building. Both here and elsewhere (Fig. 5) extensive use was made of pumped concrete. The construction programme required one bedroom floor per 12 days (two construction weeks). Despite having to bring up three conventionally shuttered lift/stair cores, a nine day cycle was averaged with a seven day minimum.

At the time of writing, the swimming pool is being cast and it is expected that the sliding roof of the pool and the topgallants of the structure will be completed by the end of April, about 21 months after starting. The standard of structural concrete work is as good as could be achieved anywhere else in the world.

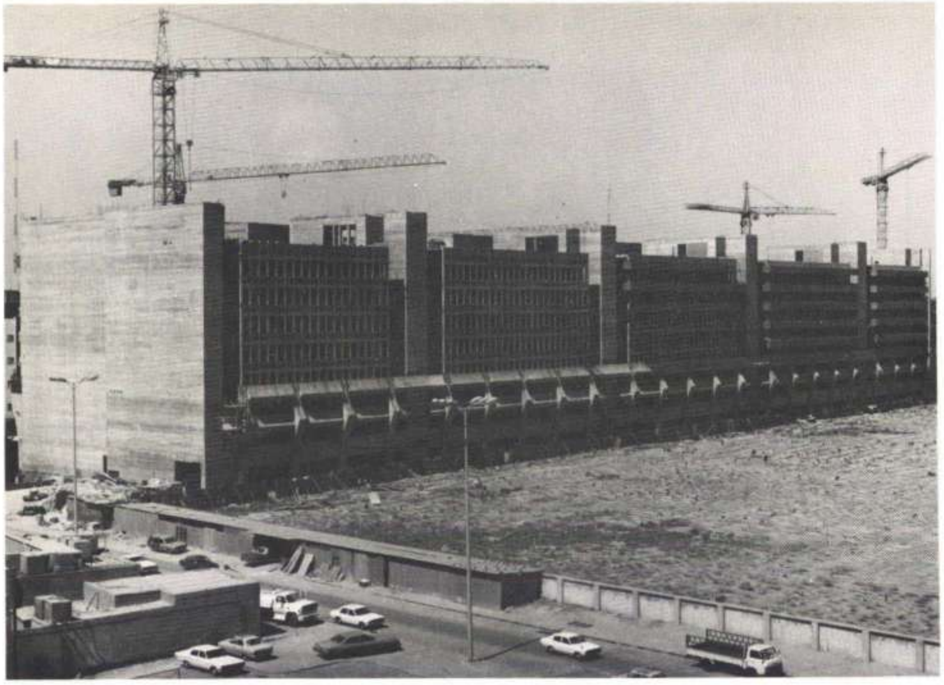


**Fig. 10**
The superstructure of the Commercial Centre at 29 September 1978 *(Photo:* Oscar Matri, Kuwait)


**Credits**

*Client:*
The Salhia Real Estate Co. SAK, Kuwait.

*Architect/Services engineer/Quantity surveyor:*
The Kuwaiti Engineering Group

*Main contractor:*
Ahmadiah Contracting & Trading Co., Kuwait

*Interior design:*
Atelier Jardonnet, Paris

*Arup resident engineers:*
Rodney Gabriel, Muhsin Dharamsi (Centre), Philip Speakman (Hotel)

Our final acknowledgement goes to the many individuals of many races and callings who contributed to this project. We hope that they will allow us to regard them as joint authors of this article.


# The strength of joints in tubular lattice towers: an investigation


Edmund Booth


### Introduction

The use of tubes in wind loaded lattice towers has many structural advantages over the use of angles, but there is one important disadvantage: the connections are much harder to make. There are two main ways of solving the connection problem: the all welded direct joint of Fig. 1 and the site-bolted gusseted joint of Fig. 2. A direct joint might well be suitable for a lightweight tower of less than 25m height which can be shop prefabricated into three or four lorry-mountable sections. The sections can be site-bolted together with the flanged connection detail shown in Fig. 3. A similar arrangement is also used in offshore flare towers where, for speedy erection, massive sections weighing over 100 tonnes can be prefabricated and shipped out to site. However, for most land-based towers, there is a limit to the size of sections that can be transported; for tall towers in remote locations this will almost certainly lead to the type of gusseted, site-bolted connection shown in Fig. 2.

This article describes an investigation into the strength of this latter type of joint, though many of the same considerations also apply to direct joints. The investigation did not break any new ground but is worth describing for two reasons. Firstly, a full scale specimen of a gusseted joint was fabricated in structural steel, covered in strain gauges and tested to failure. Thus a rare opportunity was provided to compare the results of a large finite element analysis with something rather closer to reality. Perhaps inevitably, agreement between computer and strain gauge readings was at best rather tenuous (though most published data suggest a better link up – see, for example References 1 and 2).

A second reason for describing the project is to give the engineer who is unfamiliar with the fatigue analysis of a steelwork joint a flavour of the very complex analysis involved, which despite the use of higher mathematics and hours of computer time, is still extremely crude and uncertain.

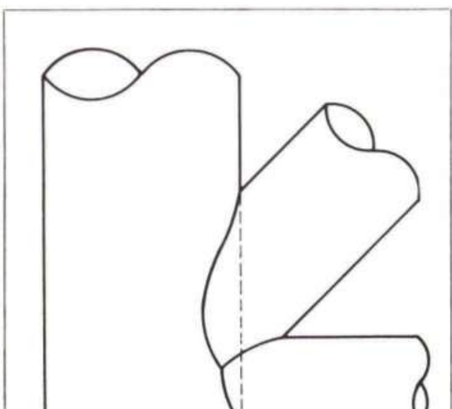


**Fig. 1**
Direct tubular joint

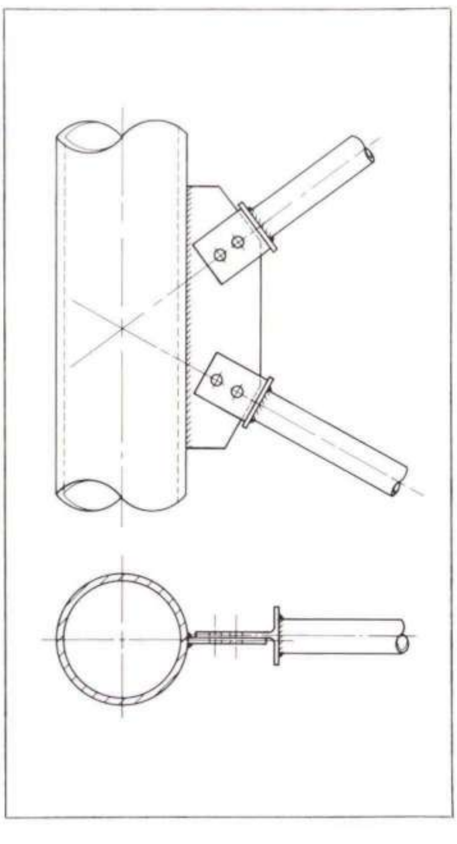


**Fig. 2**
Gusseted tubular joint

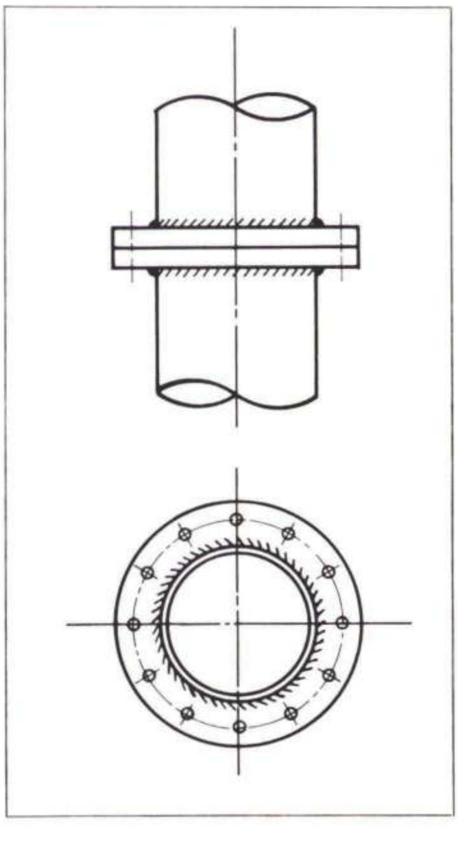


**Fig. 3**
Flanged connection detail

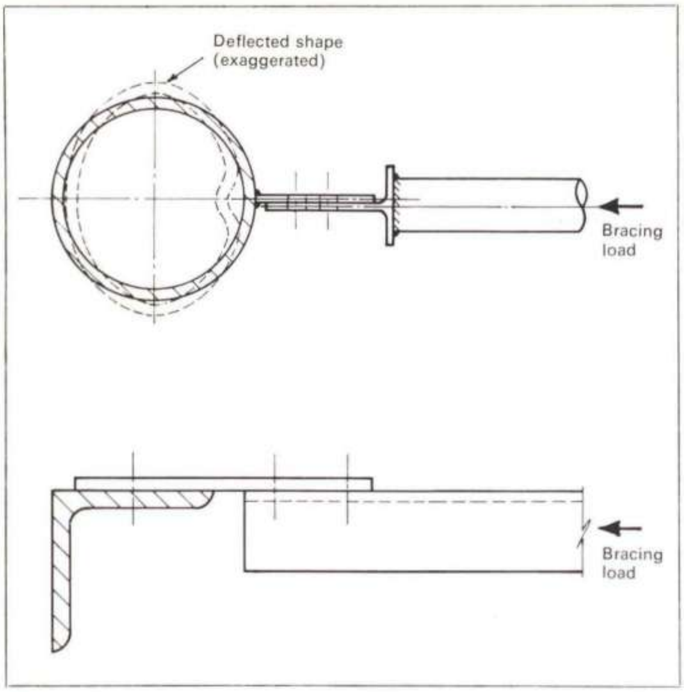



**Fig. 4**
Comparison between gusseted connections for tubes and angles

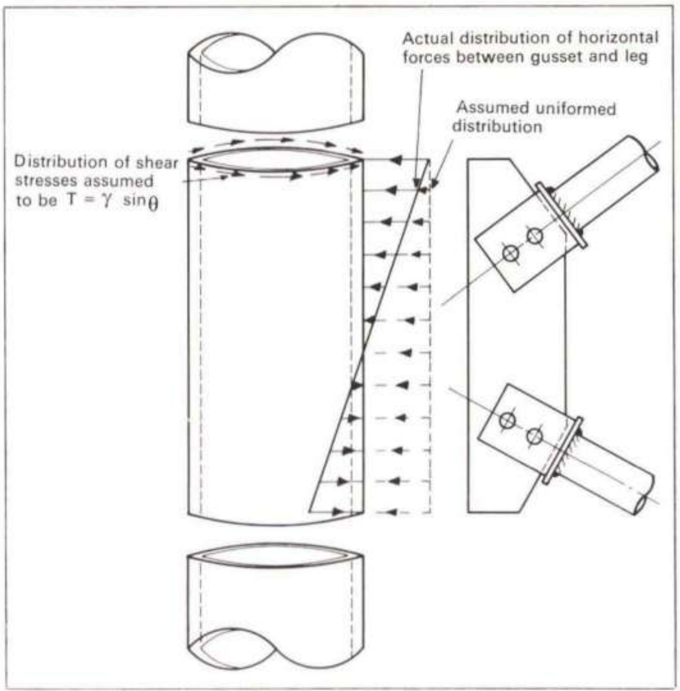



**Fig. 5**
Simplified hand analysis of stresses in tube wall

### The strength of tubular joints

It can be seen from Fig. 4 that tubular gusseted joints are by their nature more likely to be highly stressed than the equivalent joint between angle section members. In the latter, bracing forces are transferred directly into the angle leg in its stiff direction, but in the former, the bracing forces are applied at right angles to the tube wall, and so set up bending stresses in what may be very thin material.

Preliminary analysis of the gusseted joints in a typical land-based lattice tower was carried out treating the tube as an isolated cylinder (Fig. 5). This analysis gave very high stress concentrations; values up to yield stress were predicted locally when member loads were well below their design working value. The method of analysis was clearly very conservative; moreover, it is well known that plastic redistribution of stresses in tubular joints ensures load carrying capacity well after first yield (see, for example, Reference 3, page 26). However, further investigation was needed to justify the joints in two separate areas of concern; ultimate static strength and fatigue.

### The two design conditions

A typical gusseted joint in a wind loaded tubular tower may need to be checked for two conditions:

#### *(a) The static capacity*

Does a sufficient factor of safety exist against plastic 'dimpling' or perhaps buckling modes of failure under the action of member forces due to the extreme design gust? (This loading is, of course, only expected once during the life of the structure.)

#### *(b) The fatigue capacity*

Even if (a) is satisfied, the pre-yielding mode of action suggests the presence of high stresses even at quite low wind speeds. These stresses will be continuously fluctuating, due to the gusty nature of the wind, and so the possibility of fatigue failure may have to be checked. The fatigue damage is being built up continuously during the life of the joint, so one is concerned not just with extreme loads but the entire stressing history of the joint.

The aim of our project was to quantify both these conditions. The theoretical, desk-bound part of the investigation consisted of many hours of hand and computer analysis, a literature search, and consultations with experts at the Welding Institute and at the University of Manchester Institute of Science and Technology (UMIST). Experimental back-up to the analytical predictions was sought by performing a full-scale load test; there was also a site examination of some joints which had withstood eight years of service.

### Estimation of static capacity

This proved to be a relatively straightforward exercise. We assumed that the joint strength was a function of the bracing load only, and was not affected by the leg load. There appears to be no experimental data justifying this, but there are two reasons why the leg load is unlikely to change the joint strength very much.

Firstly, stresses due to the leg load are much less than the stress concentrations introduced by the bracing load and in any case are vertical, whereas the highest bracing stress concentrations are horizontal. Secondly, whilst it might be thought that a high compressive leg load would increase the risk of buckling failure of the tube wall, buckling does, however, not appear to be a problem because of the curvature of the tube wall and the stiffening effect of the gusset (see reference 3, page 27).

Empirical values of the static strength of both gusseted and direct joints are given in Reference 4 (note, incidentally, the error in line 5 of Fig 3. of that reference). These data suggest that the static strength of gusseted connections is unlikely to be a problem with the range of tube to diameter thickness ratios found in land-based towers, provided that the gusset sizes are not skimped.

### Estimation of fatigue capacity

This proved to be a much more difficult exercise. An outline of the analysis is given below. For a more general description, the reader is referred to the excellent paper by Marshall (Reference 5). A current Ove Arup & Partners Development Fund project is the preparation of specific recommendations for the fatigue analysis of wind and wave loaded structures. These are expected to be issued later this year. The Underwater Engineering Group of CIRIA also intends to prepare similar guidance on the fatigue analysis of tubular joints.

The analysis can be thought of as in four stages; the first two concern the fatigue characteristics of the welded joint in general terms; the second two stages estimate the actual loading on the joint. Each stage is now described in turn.

#### *Stage 1: Fatigue characteristics under constant amplitude loading the S–N curve*

The number of cycles at which fatigue failure occurs in a welded joint under constant amplitude loading depends principally on two factors: the magnitude of the stress fluctuation and the precise geometry of the weld – for example, whether the weld is a nicely ground profiled butt weld or an abrupt fillet weld. Other factors, such as the average level of stress, and the strength and ductility of the parent metal (assuming that is it structural steel) are much less important and are usually ignored (see Reference 6).

This neglect of average stress may surprise readers used to the old *BS 153* approach, where the ratio of maximum to minimum stress had to be taken into account. More recent research has shown that the crack-like defects at the weld toe from which fatigue cracks usually start are likely to be in a region of very high tensile prestress caused by cooling of the weld, so that the average general stress level is likely to be swamped by this prestress. Joints in which the prestress has been removed, for example by heating to a high temperature, have been shown to have a greatly improved fatigue life, especially if there is an average compressive stress present.

The higher the value of the stress range S is, the lower is the number of cycles to failure N; A plot of S against N is referred to as the S – N curve. Families of S – N curves can be drawn up corresponding to different types of weld, and these families are presented in numerous papers and codes of practice. S – N curves for design usually represent the lower bound 95% values: that is, 95% of specimens can be expected to have a longer fatigue life, but 5% will fail earlier than predicted.

Unfortunately, there is an enormous scatter in the fatigue life found experimentally for a given stress range. The fatigue life at the strongest 5% limit is found to be about 20 times greater than at the weakest 5% limit. So even at the very simplest stage – estimating fatigue loading under a precisely known, constant amplitude stress cycle – there is great uncertainty. The actual stress cycle is of course far from constant, and we shall see that the values of stress amplitude are very hard to estimate accurately.

***Stage 2: Fatigue characteristics under variable loading: Miner's rule***

The development of a fatigue crack in a welded joint is a slow but continuous process. Experimentally cracks are usually found to start at defects introduced during welding; for example, slag inclusions or heat-induced micro-cracks, and to develop through the parent, rather than the weld metal. About half the fatigue life is needed for the crack to open up to a width of 0.2 mm (probably the minimum detectable without a microscope); thereafter the crack grows at an accelerating rate until it is big enough to cause failure. This gradual building up of the crack leads to the idea of fatigue damage; mathematically, the damage ratio caused by N cycles of a stress amplitude S is defined as $N/N_f$ where $N_f$ is the number of cycles to failure at stress amplitude S, as found from the S – N curve. The damage ratio is a measure of how far a fatigue crack is down the path to failure; when the damage ratio reaches one, there is an unacceptable risk of failure. For design purposes, it is sometimes necessary to restrict the damage ratio to less than one, for example, in critical joints in inaccessible positions.

So the mathematical exercise of fatigue analysis becomes that of finding the number of years it takes to build up the fatigue damage at a joint to an unacceptable level; this period is referred to as the joint's fatigue life. The exercise is made much more difficult by the fact that a practical joint is not loaded by the constant amplitude stress cycle of the laboratory test, but by a highly variable, gusty wind. The concept of fatigue damage enables such variable loading to be handled mathematically. Suppose a joint is subject to $N_1$ cycles of a stress amplitude which from the S – N curve we know needs $(N_1)_f$ cycles to cause failure and then to $N_2$ cycles of a stress corresponding to $(N_2)_f$ cycles to failure, . . . and so on; the cumulative fatigue damage is defined as

$$\text{Cumulative damage} = \sum_{r=1}^{n} \frac{Nr}{(Nr)_f}$$

Each part of the loading develops the crack slightly more, and one might expect that failure finally occurs when the accumulated damage reaches one. This is what Miner's rule states; there is experimental evidence to show that it is usually conservative, (see Reference 7) though the main reason for its use is that any less simple assumption would make analysis impossibly complex. Miner's law assumes that the order of loading doesn't affect the fatigue strength – that it makes no difference whether a low amplitude cycle proceeds or follows a high amplitude one. There is, however, ample experimental evidence that the order of loading does make a difference – though it is harder to say whether it is better for a very strong wind to occur at the beginning or end of a structure's life. The typical laboratory verification of Miner's rule also uses a much simpler loading spectrum than the highly complex one a practical joint will experience. Presumably, a spectacular series of failures in the North Sea would tell us if it was unwise to trust that this complexity does not make matters worse.

Miner's rule then allows the data for constant amplitude loading – the S-N curve – to be applied to variable amplitude loads,

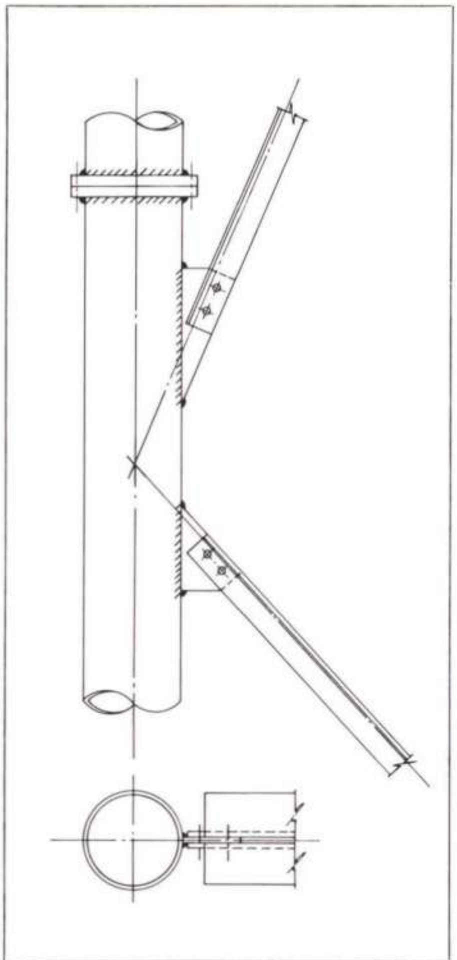


**Fig. 6**
Details of joint subjected to finite element analysis and full-scale load test

though even if those loads are known with certainty, there will be great uncertainty in the prediction of when fatigue failure will occur. In principle, however, if the number of cycles of each amplitude of stress can be found, an estimate, however uncertain, of fatigue life can be made. The next requirement, therefore, is to predict the joint's loading history. This is done in two stages – finding the peak stress caused by a given constant bracing load, and then estimating how that bracing load varied with time. These two stages are now described.

***Stage 3: Calculation of peak stress concentrations in the joint***

The action of member loads on a joint causes a highly complex system of stresses to be set up in the region of the joint. The requirement for the fatigue analysis is to find the peak stress due to a given bracing load at the toe of the joint weld, since this is the critical region. No account should be taken of the stress raising effect of the weld profile itself or of residual stresses introduced by the welding process, since these are already allowed for in the S – N curves described in Stage 1.

A great deal of literature covers the evaluation of stress concentrations in direct tube to tube joints (see, for example, References 1 and 2) but simple gusseted joints are seldom used in offshore applications and so have not been much investigated. In our investigation, therefore, we had little published guidance. One of the joints we analyzed is shown in Fig. 6; we first subjected it to the simple hand analysis of Fig. 5, using Table 17, cases 13 and 19 of Roark (Reference 8). This analysis did not take account of the beneficial effect of the elastic spread of stresses beyond the gusset plate, or of the stiffening effect of the leg flange connection plates. The arrangement in Fig. 6 was subjected to a finite element analysis which did allow for these effects and managed to reduce the stresses by a satisfying factor of three compared with the hand analysis.

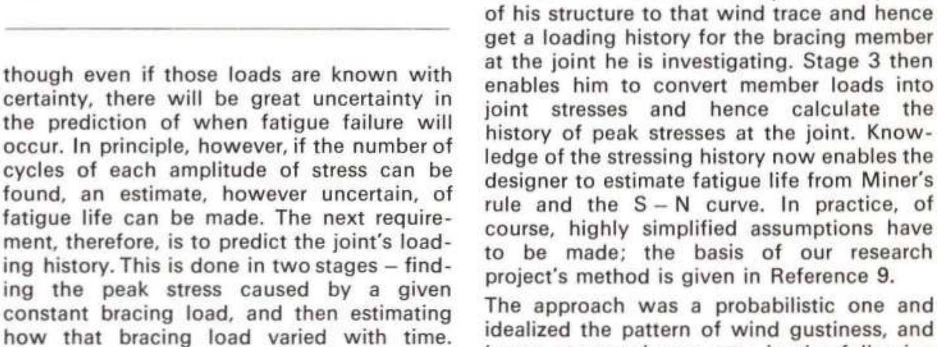
Still, despite the tediousness and cost of the finite element analysis, it could hardly be described as accurate. It failed to take into account such factors as rolling tolerances in the tube wall thickness, fabrication tolerances in gusset size, effect of leg loads on stress concentrations and the finite thickness of the gusset plate and weld. It can therefore be seen that this stage of the stressing analysis – calculation of peak stresses due to a given bracing load – is not only time-consuming and expensive but also rather uncertain.

***Stage 4: Estimation of loading history***

The uncertainties of the previous stage pale before those of this next one. The designer has to guess what winds will be blowing during the entire design life of his structure. He then has to estimate the dynamic response of his structure to that wind trace and hence get a loading history for the bracing member at the joint he is investigating. Stage 3 then enables him to convert member loads into joint stresses and hence calculate the history of peak stresses at the joint. Knowledge of the stressing history now enables the designer to estimate fatigue life from Miner's rule and the S – N curve. In practice, of course, highly simplified assumptions have to be made; the basis of our research project's method is given in Reference 9.

The approach was a probabilistic one and idealized the pattern of wind gustiness, and hence structural response, in the following way. Both excitation and response can be split into a mean component, averaged over an hour, plus a random fluctuating component – see Fig. 7. These random fluctuations were assumed to follow a 'Rayleigh distribution' (see Fig. 8). It can be shown mathematically that the fatigue damage caused by a Rayleigh distributed stress spectrum depends only on the frequency and RMS (root mean square)

**Fig. 7**
Mean hourly and fluctuating components



value of stress fluctuation – and also on the S – N curve described in Stage 1. Relatively simple mathematical expressions can be derived to relate these parameters to fatigue damage.

Ove Arup & Partners have a dynamic analysis computer program which is well equipped to calculate frequency and RMS values. Given values of mean hourly wind speed, ground roughness conditions and the tower's damping, the program will output the RMS values of member force fluctuations directly.

We were now in a position to piece all the parts of the analysis together. A typical land-based tower was chosen, and subjected to the dynamic computer analysis under the action of the extreme design wind speed; this wind speed had already been found for the tower's static analysis. The RMS fluctuations so obtained could then be used to calculate the fatigue damage caused by the extreme wind blowing for an hour. The smaller damage due to an hour of a lower wind speed was calculated by quite simple ratio methods. We now knew the fatigue damage caused by an hour of any wind speed. If we could find how many hours each wind speed would blow during the life of the structure, we could then easily work out the total fatigue damage during the design life from Miner's rule:

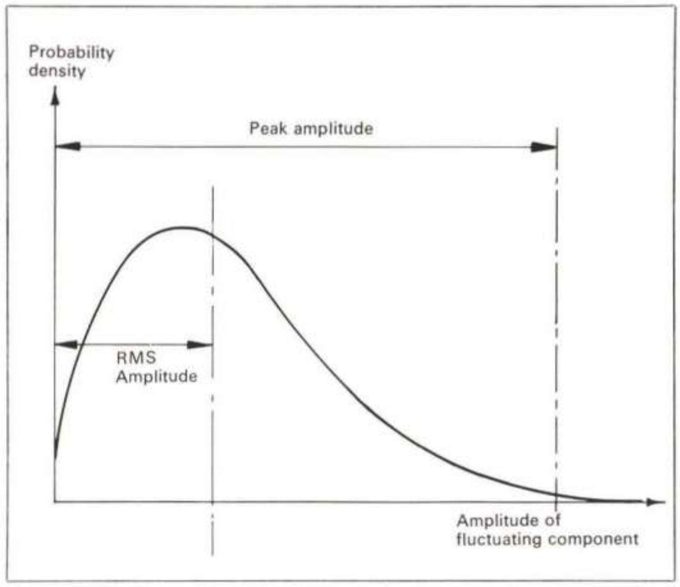
$$\text{Total fatigue damage} = \sum \begin{pmatrix}\text{one hour's damage}\\ \text{caused by a wind speed}\end{pmatrix} \times \begin{pmatrix}\text{number of hours that}\\ \text{wind speed blows for}\end{pmatrix}$$

In practice, the calculation was carried out for a set of wind speed ranges (say 0–10 kph, 10–20 kph and so on). The problem, of course, was to find out how many hours of each wind speed range could be expected. Even in a country with good wind records like England, these data are not easy to find, since, in the past, meteorologists have been more interested in extreme values than intermediate ones. In remote locations in developing countries, the problems of finding adequate data are even worse.

Taken by itself, this final stage of the analysis – estimating the loading history of the joint – had many uncertainties. Principally, these lay in finding the values of structural and aerodynamic damping of the structure (for which Reference 10 gives some guidance) and the probability distribution of wind speeds. These uncertainties were compounded with those of the previous stages, which were, to say the least, massive.

It is hoped that this description has shown that it is possible to make a quantitative assessment of fatigue life, and that in addition some of the pitfalls and approximations in the assumptions for the analysis have been made clear.

**Fig. 8**
Rayleigh distribution




### *Comparison between theory and reality – the full-scale load test and site investigation*

Because the theoretical analysis was so uncertain, two practical strength tests were carried out. The first was performed by Professor Burdekin of UMIST, and involved the testing of a full-scale specimen of the gusseted joint shown in Fig. 6, made out of welded structural steel. The test arrangement is illustrated in Fig. 9. The test was intended to check the stress concentrations predicted by the finite element analysis and also the static strength of the joint. It was not intended to test the fatigue capacity directly. Loading was applied to the top brace; restraint to the bottom of the leg and the lower bracing member was arranged to ensure that all members were axially loaded. The top of the leg member was left unrestrained. Twelve electric strain gauges were stuck to the tube wall of the leg, and the reaction at the bottom of the leg was measured directly by three load cells.

The load was increased in 5 kN increments up to the onset of plastic behaviour, and the corresponding strain gauge readings were noted. After yielding, loading was increased more rapidly until failure occurred.

Comparison between predicted and recorded stresses in the elastic range was poor. In all cases, the recorded stresses were lower than expected. This may have been due to the high rate of change of stress compared with the length of strain gauge; the effects of residual welding stresses are also difficult to evaluate. Great care had been taken in choosing the finite element size, but it is possible that further refining the element mesh would have resulted in better agreement. Other researchers seem to have more success in their comparisons; see, for example, References 1 and 2. The only crumb of comfort was that non-linear behaviour, as recorded by the strain gauges, started at within 10% of the load at which the computer predicted yield stresses would be reached.

Failure finally occurred at a load 50% higher than that predicted by Reference 4, by shear in the bolts connecting the top bracing member to the gusset. After removal of the load, no sign of distortion of the tube wall around the gusset could be found, so the static strength rules of Reference 4 seem to be well on the safe side, for this case at any rate. (Incidentally, the bolts which finally sheared had been predicted to fail at a load some 30% lower than they actually did, which perhaps puts the static strength rules into some perspective.)

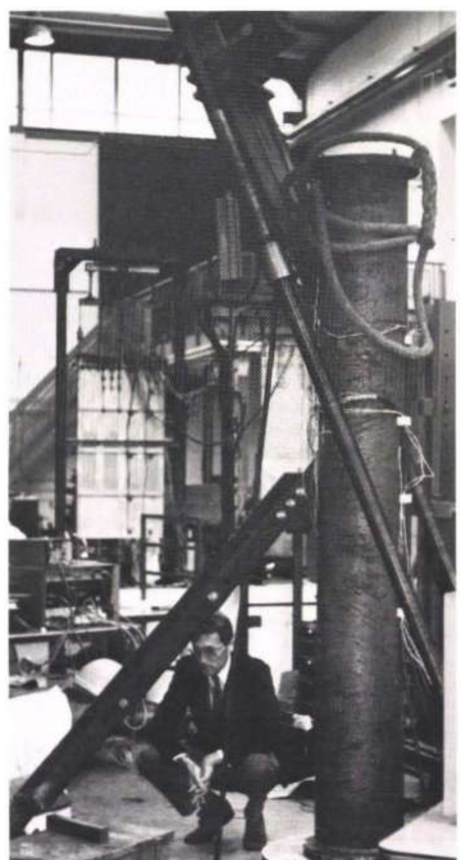
The second practical test was a site investigation, mainly looking for fatigue cracks using dye penetrant analysis. 50 critical gusseted joints were inspected on some land-based win loaded lattice towers which had seen about eight years of service. No fatigue cracks, nor any other signs of distress, were found. This was in general confirmation of the theoretical analysis, which had predicted that fatigue cracks were unlikely, and that static capacity was adequate.

### Conclusions

We have seen that fatigue analysis is at best an uncertain exercise. However, the mathematical analysis and site investigation did allow some general conclusions to be drawn. Provided an adequate factor of safety on static capacity exists in all joints (a cautious minimum factor for grade 43 or 50 steel would be 2), fatigue failure is unlikely to occur in the connections of a typical land-based, wind-loaded tower, and need not be checked, except in special circumstances (for example, where such a check is required by a certifying authority). This conclusion applies even to towers with all welded joints. Bolted connections greatly increase the structural damping, and so reduce the fatigue risk still further.

Fatigue problems are much more likely to occur in wave-loaded structures where fatigue checks will almost certainly be necessary. This article may have given the impression that the mathematical analysis of fatigue strength is so uncertain that it is not even worth carrying out. However, the exercise can have considerable value, provided that its limitations are

**Fig. 9**
UMIST test arrangement
(*Photo:* Edmund Booth)



understood. If a joint in a structure with a 20-year design life has a fatigue life calculated to be at least 50 years, it is definitely satisfactory, whereas a joint with only a 1-year life is definitely unsatisfactory. Moreover, a fatigue analysis will help to ensure that the fatigue strengths of all joints are well matched, so that some joints do not have strengths very much less than the general level; it will also highlight the areas where special measures may be required, for example grinding or peening of welds, periodic in service inspection for cracks, stress relieving and so on. It remains true, however, that the most important aim of fatigue design is not to ensure that the calculated fatigue life is greater than some arbitrary figure, but, by means of good geometrical detailing of joints, to avoid the local areas of high stress concentration ('hot spots' in the jargon) that give rise to fatigue failure.

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# Abu Dhabi Trade Centre

**Architects: Scott Brownrigg & Turner**


David Lewis


### Introduction

In April 1975 Ove Arup & Partners were invited by the architects Scott Brownrigg & Turner to join their design team of quantity surveyors, Widnell & Trollope, and M & E consultants, John Bradley Associates, for the design of a proposed Abu Dhabi Trade Centre. The client, Sheik Suroor Bin Mohammed Al Nahyan, requested several scheme designs with sites at various locations on the island until he was satisfied that the building was of the required form and economically viable. This was confirmed by a feasibility study undertaken by estate agents Debenham Tueson & Clark.

The long-standing project finally began on site in October 1977, and is now nearly structurally complete. Initial scheme designs ranged from two to 40 storey buildings with and without multi-storey car parks, but the project as built comprises two seven-storey office blocks with provision for shops on the ground floor and a two-storey supermarket block incorporating a cinema and coffee shop.

Originally the tender documents were in the form of standard FIDIC conditions with a bill of approximate quantities but in March 1977 the client stated that he was unhappy with this arrangement. He expressed his wish to go out to tender only eight weeks later, on full working drawings and specifications, with an abbreviated bill of quantities. This was achieved at great effort by the design team and the National Construction Company (Pakistan) Ltd. were the successful tenderers. The contract period was to be approximately 18 months since the client required the building to be serviceable as soon as possible. The construction is somewhat behind programme primarily because NCC, whilst consistently producing work of a very high standard, has been less rigorous in the forward planning, ordering of materials and co-ordination of subcontractors that is required for a job of this nature.

### Foundations

The ground conditions are typical of an offshore island adjacent to the Gulf mainland. The bedrock is sandstone at a depth of approximately 5m, overlain by loose marine deposited Oolitic sand topped by approximately 1.5m of pumped compacted sand fill used to reclaim the land from the sea. The ground water level is about 1.8m down and fluctuates with the tide. During the initial site investigation, cavities were detected in a narrow band of Dolomitic limestone about 20m down formed long ago by percolating water leaching out soluble gypsum.

Virtually all buildings in Abu Dhabi are supported on piles driven into the bedrock but, due to the cavities, this form of foundation was considered unacceptable for the Trade Centre. In order to keep expense to a minimum and avoid considerable dewatering, foundations were designed as individual pad footings above the ground water level. This scheme required improvement of the allowable bearing capacity of the Oolitic sand. The technique of *Vibroflotation* was used to achieve this requirement. Approximately 2500 compactions were made on the site which was then subjected to a plate loading and dutch cone testing programme. This revealed that a 150mm thick layer of silt, forming the original seabed and just under the founding level, had not been washed out and dispersed sufficiently. The subcontractor placed a further 2000 compaction points through this silt in order to guarantee the acceptability of the ground. This proved satisfactory and subsequent tests showed that the foundations could be constructed at a depth of 0.8m and safe bearing pressure of 300 kN/m².

Prior to commencing construction on site a building licence was required from the local municipality engineers who unfortunately took a rather pessimistic view of the success of the ground improvement scheme. All attempts to satisfy them that the seven-storey buildings

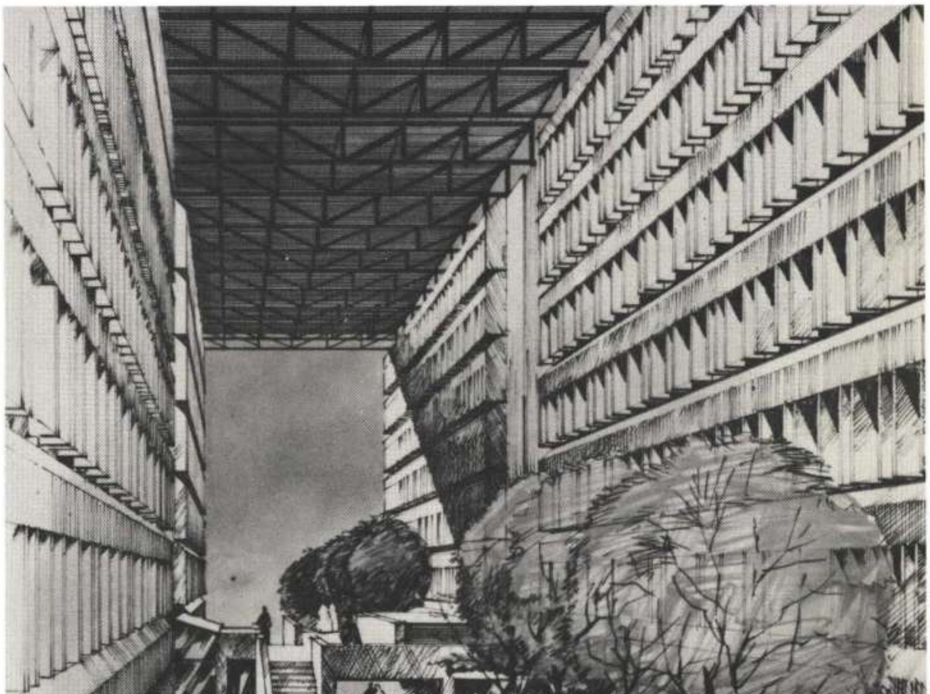


**Fig. 1**
Artist's impression of shaded garden
(Illustration reproduced by courtesy of Scott Brownrigg & Turner)

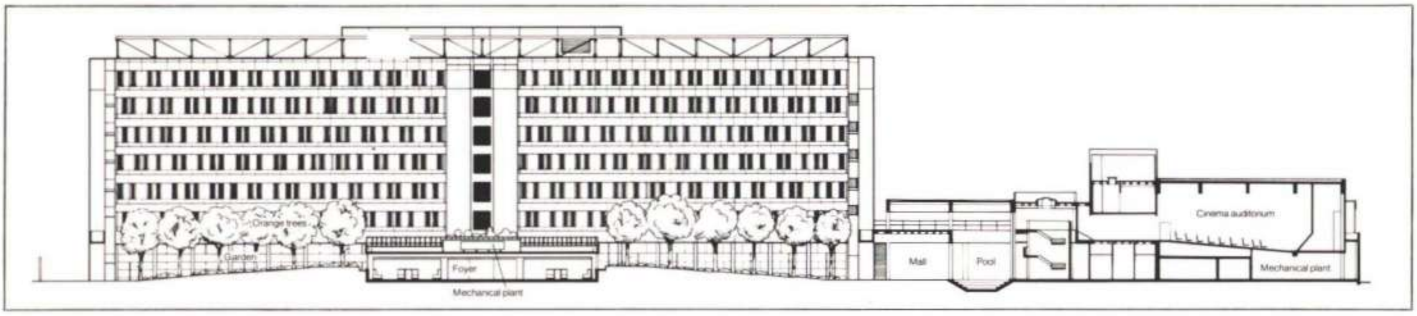
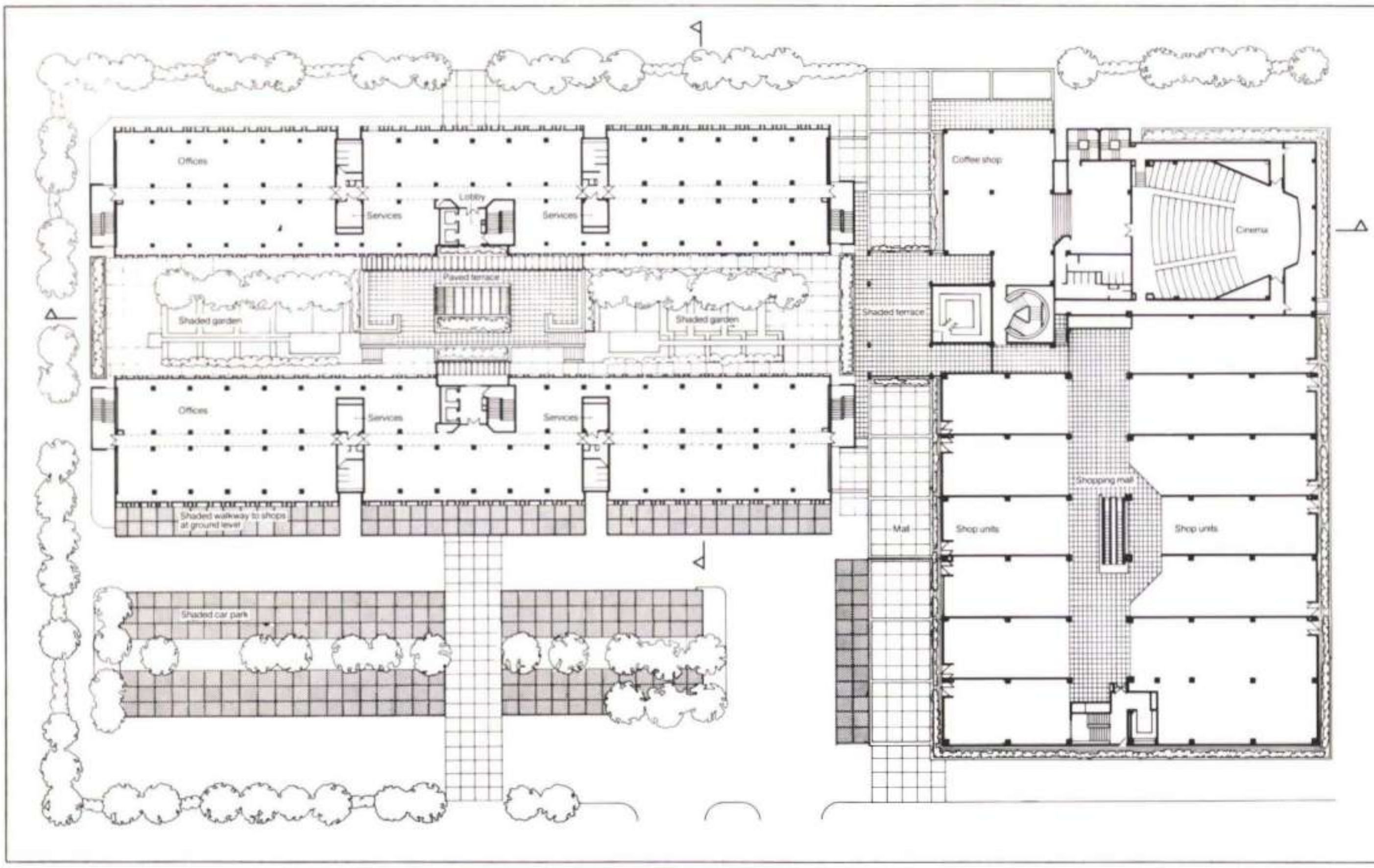



**Fig. 2**
Longitudinal section

**Fig. 3**
Plan at first floor level




were safe with the designed pad footings failed and, as a result, the resident site staff redesigned the foundations in the form of a semi-cellular raft system with a bearing pressure of 150 kN/m². Pad footings for the two-storey building were retained but increased in size in order to conform to the reduced bearing pressure also.

**Structure**

The office blocks are formed of in situ concrete slabs 200mm deep which span 4.5m onto transverse beam and column frames at the front and rear of the 14.5m wide blocks. This leaves a 2m beam-free corridor between the two central columns for the services distribution. Stability of the blocks is provided by concrete walls forming stair, toilets and lift cores. The 90m long blocks are parallel, forming a protected mall 15m wide between them. The main entrance foyer is situated at the centre of this area with access provided through the front block. At roof level of the office blocks a shading lattice covers the mall. This is formed from screens of diagonally crossed timbers giving a solidity ratio of approximately 75%. They are supported by large timber trusses at 4.5m centres and spanning 17m onto the roofs of the blocks. These buildings are clad with precast concrete panels.

The supermarket block is formed of 400mm in situ concrete waffle slabs on columns at 7.5m centres. Stability of the building is provided by frame action between the columns and slabs. Access is provided to the first floor shopping area by escalators, some of the first to be incorporated in a building in Abu Dhabi. All three buildings have structural movement joints at 30m maximum centres. These are formed in the office blocks by a sliding bearing support system and in the supermarket by multiple column arrangements.

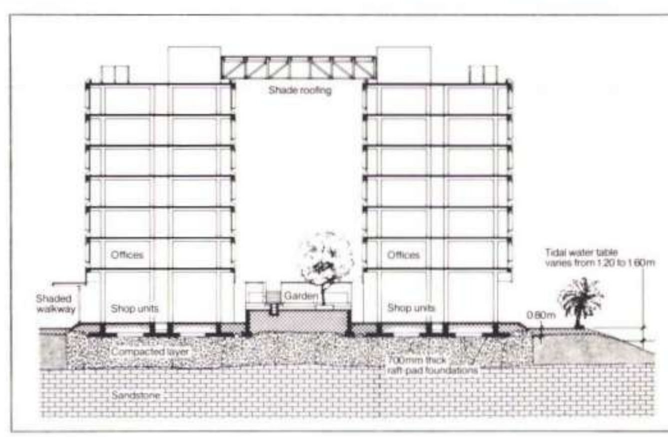



**Fig. 4**
Transverse section

**Materials**

Supplies of materials from consistent sources are impossible to guarantee for an overseas project of this nature. Even separate batches from the same source can have different characteristics. All materials used in the structure were therefore subjected to a rigorous independent testing programme. This ensured that compliance with the specification was achieved and the serviceability requirements for the structure could be guaranteed in the harsh environment of the Gulf area. The effects of a humid and salt laden atmosphere can quite easily be seen in many poorly constructed buildings throughout the city.

All reinforcement originated from Japan and was of a very high quality. Initially cement was also Japanese but as supplies became less available, European and finally locally produced cements were introduced to the job. Suitable screened aggregates were obtained from wadis approximately 150 miles inland by the Oman–United Arab Emirates border.

Fig. 5
General view of site
(*Photo:* Bob Walster)

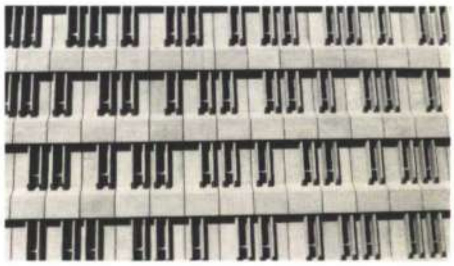


Fig. 6
Detail of cladding panels
(*Photo:* Bob Walster)

Fig. 7
View looking north
(*Photo:* Bob Walster)

Fig. 8
Front of office buildings viewed from supermarket block
(*Photo:* Bob Walster)

**Construction**

The contractor employed three tower and two mobile cranes in the execution of the contract. Two tower cranes were located at fixed positions between the office blocks serving all construction activities for these buildings. The third tower crane was able to travel on caterpillar tracks to serve the supermarket block.

It was critical that construction of the office blocks should be achieved as quickly as possible. The various trades of the labour force worked as separate teams rotating around these buildings. The preparation of each area was achieved rapidly to ensure that slabs and vertical structure could be poured almost continuously. At times, when office construction had not progressed sufficiently, placing of concrete was switched to the less critical supermarket block where work had been prepared in anticipation of this situation. Regularly, concrete was being produced and placed throughout the entire working day of about 12 hours.

All concrete was batch-produced on site using a 0.5 $m^3$ semi-automatic pan mixer. This enabled quality control of concrete to be monitored directly by the resident staff. Mix design incorporated a plasticizer additive allowing good workability with low water cement ratios, producing concrete of a quality that was entirely acceptable.

All reinforcing steel was cut and bent on site, working from schedules produced by the contractor using details on our working drawings. These were thoroughly checked by the resident staff prior to use and consequently no major problems were experienced in fixing steel. Setting out, shuttering and steel fixing were generally very good.

Curing of slabs was achieved by spraying freshly placed concrete immediately with a curing membrane followed by damp hessian when the surface had sufficiently firmed. This hessian was kept wet for several days ensuring adequate curing of the concrete. Slabs were poured in the late afternoon, when air temperatures had reduced, allowing shuttering and reinforcement to be cooled down by the evaporation of sprayed water. This also had the advantage that initial curing took place during the cooler nights. Shuttering for vertical structure made these items less critical and they were poured during the early morning.

Even though slab areas were quite large, in order to maintain the programme, shrinkage cracking was almost eliminated using these curing techniques. Some cracks occasionally developed due to plastic settlement around the top steel. These were successfully removed by the contractor retrowelling the surface just prior to the concrete going off.

**Precast work**

The site was insufficient to allow the formation of a precasting yard and panel storage area. The main contractor had to subcontract this aspect of the job. Great difficulty was experienced in finding a suitable subcontractor who could undertake the scope of precast panel production with the high degree of finish required.

On our first visit to the chosen precasting yard we saw panels being produced using concrete of very low quality in dirty moulds too hot to touch.

In order to produce panels for the Trade Centre the subcontractor had to comply with our specification requirements and commenced using the same materials and concrete mixes as on the main works. As a result of this all aspects of his work in the precasting yard improved.

Steel moulds (of very high quality) were obtained from Europe and trial panels were produced in September 1978. Basic initial problems such as demoulding, minor grout loss and uniformity of colour took some 20 trial panels to overcome until the resident staff were confident that satisfactory panels were being produced. Completion of the project was delayed by production of the precast elements.

**Summary**

As our first major structural project in the United Arab Emirates this job may introduce Ove Arup & Partners to further work in this Middle East Country. Compared with the majority of other buildings in Abu Dhabi, we consider that the Trade Centre will stand out as an example of good structural design and quality construction. We are hopeful that this project will provide a basis for our reputation in this area of the Gulf for providing engineering services of a high standard.

**Credits:**

*Client:*
Sheik Suroor Bin Mohammed Al Nahyan
*Architect:*
Scott Brownrigg & Turner
*M & E engineers:*
John Bradley Associates
*Quantity surveyor:*
Widnell & Trollope
*Main contractor:*
National Construction Co. (Pakistan) Ltd.
*Ground improvement:*
Foundation Techniques

# Building services design in the context of security

David Lush

### INTRODUCTION

The complete range of activities in the fields of violence, vandalism and terrorism, which may affect the design of building services from a security point of view is enormous. The intention is to provide an overview of the situation in an attempt to cover as wide a range as possible, using some specific examples. The examples spread into associated design disciplines because it is essential that design professions overlap and communicate, particularly at their interfaces.

It may appear simple to separate the effects of vandalism, violence and terrorism on building services into separate compartments. In practice, Building Services Security is the most comprehensive general heading as it covers acts of vandalism and terrorism. The division between what vandals and terrorists can achieve in relation to some building services is a very grey area and the worst excesses of vandalism are no different from some terrorism.

In security terms, to cover the terrorism aspects, many of the points covered relate to very high security risk buildings, to suit the most complex situations. There is no suggestion that this is necessary for all buildings, nor would we want to live in such a society. However, these problems do arise and the level of security has to be evaluated against the risks. The necessary precautionary measures for the risk should, whenever possible, be part of the initial building design.

#### Vandalism

Vandalism is defined as 'wilful destruction', but what motivates the vandals? The physical environment of the building, its design, detailing and fittings may encourage vandalism. It cannot be condoned, but the effect of entering gloomy lift lobbies, depressing subways and walkways and unsupervised stark toilet facilities is depressing, even to the well-adjusted person. Under such circumstances light fittings, toilet accessories and lifts are magnets to the vandals who have time on their hands, no outside interests, and in such surroundings may well be encouraged in their destructive efforts by their friends. It should be remembered that access cannot easily or properly be denied to areas such as those described above, because the majority of such buildings and areas are part of the public domain.

What can be done? In the long term, better education should be the answer. In the shorter term there are various remedies. The first is to select fittings, lights in particular, which are classified as vandal proof and which can also be recessed wherever possible. The term 'vandal proof' has only a limited meaning and care has to be taken to select the most suitable from those available. Integrated design is necessary, even for this small element, as the architect and structural engineer have to permit the fittings and conduit to be completely carcassed into the building fabric. A second remedy is to ensure that such areas in buildings are less depressing and better lit; this affects planning, architecture and cost.

Both solutions carry an initial financial penalty but this will normally prove the cheaper option when applied to cost in use over the life of the building.

This is a very generalized statement about vandalism and specific examples are detailed elsewhere. There is no universal panacea in design terms that will stop all vandalism.

#### Violence

If considered by its definition 'as the unlawful exercise of physical force' then violence is not related directly to the topic of building services. Certainly most vandalism is based on violence but the more general aspect relates to violence external to buildings. The brief here is to cover building services which, by normal standards, are those within the building or external but associated with the building complex. Street-mounted services equipment is outside the brief but mob violence can wreck what are normally considered to be very substantial pieces of equipment, in order to use them as weapons or obstacles. The solution to this lies in the maintenance of law and order within the community as a whole.

### BUILDING SERVICES SECURITY

#### General

When acts of terrorism rather than simple vandalism are related to building services a much more complex problem arises. Indiscriminate terrorist bombing against all types of building requires a decision as to whether all buildings and building services need to be stiffened against explosions. If Beirut and Northern Ireland are taken as examples of this, the answer must surely be in the negative as there can be no economic justification for such action. Nonetheless in such situations personal and building search facilities and fire precautions need to be generally upgraded and the dangers from shattered glazing should be minimized.

The more common examples, even if less in number, can be described as isolated terrorist attacks carried out under different banners for a variety of reasons. The philosophy behind this is not part of this survey. The terrorists use fear and blackmail as their allies, based on bombing buildings, occupation of buildings and the holding of hostages or combinations of these. The only sensible direct attack on building services, viewed from the terrorist's point of view, would be on those serving communications centres, computer suites and, possibly, government or industrial process plants of secret or dangerous products. One can philosophize on whether the terror and fear generated when an explosion occurs visibly is of greater use to the terrorist than disabling a building for a considerable period by destroying its services with no visual impact. This however is outside the scope of this review but should be considered as a possible future development.

(1) Damage and destruction by explosion
(2) Access via building services systems
(3) Cutting off essential external supplies
(4) Introduction of harmful substances
(5) Specific problems and possible remedies

**Fig. 1 above**
Major considerations for building services security

Petrochemical and nuclear plants require special consideration, although many of the general principles for building services security will still be applicable. Recent instances of explosive devices at gas works and oil refineries illustrate this and emphasize the necessity to restrict access to sensitive areas.

The first decision which must be taken therefore, is to select which buildings or building types need to be designed to withstand such attacks. The examples already quoted cover the broad range of likely possibilities but the final choice will be governed by a number of factors which are outside the technical brief of this paper.

A check list such as that produced by certain UK police authorities should be used as the basis for selection generally, with suitable modifications to cover the building services requirements.

Having decided which buildings need such treatment the actual design of building services to withstand terrorist attacks may be considered under several basic elements. Some of them, listed in Fig. 1, also relate to the problems of vandalism.

The means by which the design brief for security purposes is developed is a function of the design team, in accordance with their specialized knowledge, applied to the client's building requirements. The inter-relationship between the various involved parties is shown in Fig. 2.

#### Designers and their selection

Before the elements of Fig. 1 are treated in turn, consider the method of design to be

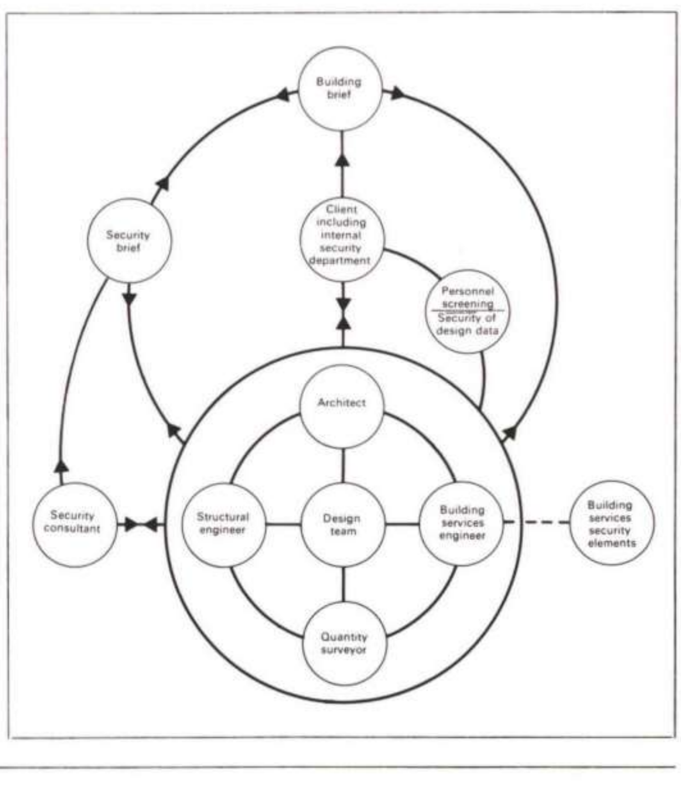



**Fig. 2**
Relationship between client's requirements and design disciplines

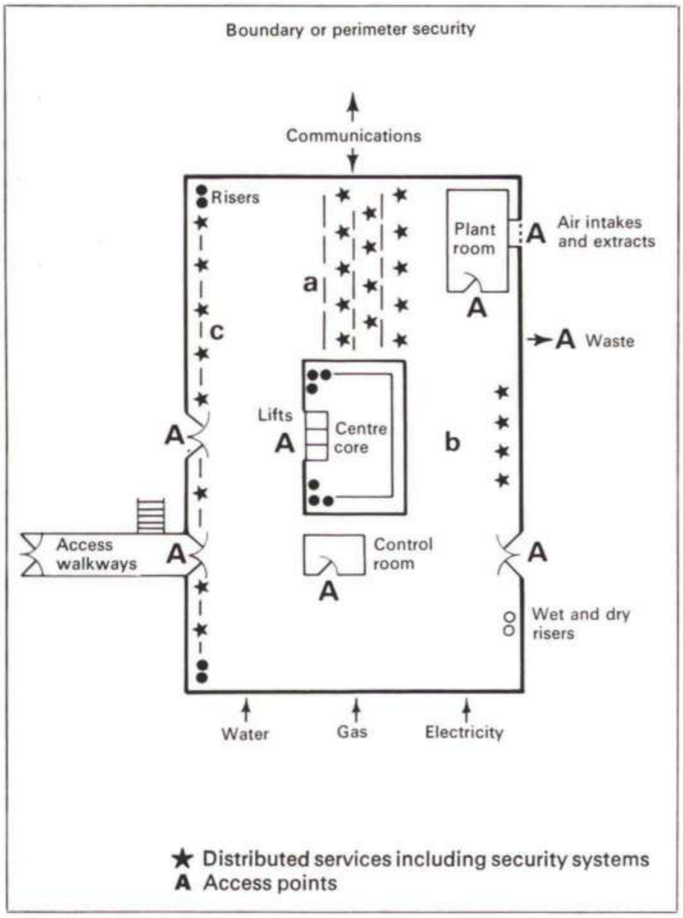



**Fig. 3**
Building services security elements affecting design

adopted. Whether the problem is one of restricted access, damage directly or indirectly from explosives, or any of the other elements, the subject of co-ordinated or integrated design should be considered. Architects, structural engineers, building services engineers and quantity surveyors working together rather than independently should, in normal circumstances, be able to produce an integrated design with improvements over a similar, conventionally designed building. In the special circumstances of vandalism and terrorism such integration would seem to be even more important. Apart from the technicalities of the design which will have innumerable interface conditions for security purposes, the cost implications of certain solutions need immediate consideration.

Designers from separate professional disciplines can of course offer acceptable solutions for the various security aspects of building design but integration will improve many facets of the design.

Fig. 2 illustrates how the designers should coordinate their efforts and the role of security consultant may be filled by the client, police, specialist consultant, the building services consultant or a combination of these, depending on circumstances.

One other point which touches on the design team, however it is made up, is the possible necessity for security screening. This is dependent on the type of building and the level of security required, but it may be essential in certain cases. Design data of certain elements of building structure, services equipment and their siting within the building would be very useful to organized terrorist groups. Thus design information may also need to be kept under secure conditions and this creates an additional range of problems.

The problems fall into two basic categories. First, an indexing and distribution system has to be applied so that data is available on a limited and recorded scale on the basis of a 'need to know' and it must be totally recoverable. With the number of sub-contractors and suppliers on a conventional building with innumerable interface situations this is a major technical/administrative task in its own right which may be a costly extra on the project. Secondly, when the project is finally in operation and the data and drawings have to be used around the complex for servicing and maintenance purposes, a similar set of circumstances occur over the full life cycle of the building.

### Explosion damage and siting of distributed services

Returning to the basic elements of Fig. 1, consider first the explosion risk. Explosive destruction to building services in the past has been caused indirectly. Charges set to damage the fabric and structure, while causing terror and panic at the same time, have damaged services either by blast or collapsing builders' work. To date, very few specific explosive attacks have taken place on building services installations internally. Of the two known to the writer, one in a service shaft could have been a random positioning, while the other in a telephone exchange appears to have been deliberate and may illustrate the thinking for more sophisticated attacks in future.

If it is accepted that such attacks are to be directly aimed in future at the services, within the building itself, then there is no acceptable solution in design or cost terms. Basically the answer must be to prevent access, first to the building and then to the main plant areas. Designing ductwork, pipework and most plant against adjacent explosions is not really feasible.

Without specifying which types of building and usage are likely to be subject to terrorist attacks, one or two general design principles may be suggested. Fig. 3 illustrates the considerations for various building services, access points, positioning and distribution.

Where the building is designed for maximum security, i.e. little or no glazing, it will undoubtedly be air-conditioned. In such circumstances, design the services distribution so that it is secured and distributed from the core structure rather than the perimeter (Fig. 3(a)).

Whether the structure is then proof against explosive charges or not, the effect on the building services will be minimized. In extreme conditions use completely independent plants to serve separate sections of the building.

Other types of building which can be identified as terrorist-prone, but of more conventional construction, will presumably still require some perimeter heating (or cooling) system. Where possible this should be designed so as to provide the minimum possible contact with the perimeter structure (Fig. 3(b)), rather than the normal distribution (Fig. 3(c)). Risers which are conventionally carried adjacent to the perimeter should be repositioned in protected areas such as the central core.

Similar principles should be applied for water, air and electrical distribution systems. The risk of explosion damage to services is one which simply emphasizes the recurrent theme of limiting access to the buildings and their services. A competent terrorist, once he has entered the building can, with minimal knowledge and a small amount of explosive, disable a conventional building by wrecking the services.

### Infiltration into buildings

This is the second element of Fig. 1. Access to buildings through doors or windows is reasonably easily monitored for security purposes, the level of security required governing the sophistication of the system employed. Various other means of access may also exist through certain of the building services. Fig. 3 indicates these access points. Such access needs to be restricted, first by means of proper design and secondly by suitable security measures and monitoring.

The building service which offers the best opportunity for unauthorized access is the ventilation or air-conditioning plant. Almost without exception any ductwork system serving such a plant will be large enough for access. Equally, most buildings with such intake or exhaust systems have at least some of them virtually at ground level. Once past the first decorative louvred panel at the access point, the infiltrator, with the panel replaced, can gain entry at his leisure.

Where such low level access exists the first priority must be to make it secure by monitoring it as part of the general alarm system. When buildings are being specifically designed for security purposes these access points should be removed from ground level to inaccessible, intermediate or roof levels, facing inwards if the building configuration permits. To do this on a large complex may require a major change in the design philosophy and certainly dictates that all the design disciplines are involved together at the concept stage of the design. Another reason for this feature will be apparent under one of the later elements. Even with the correct siting of these intakes they should be included in the security alarm system in the same way as conventional entrances.

One other building service which may permit unauthorized and illicit entry is that of public health or drainage. Dependent on the geographic location and methods of waste disposal there may be means of access available from both the rainwater and foul drainage systems. Such access will be to basement or ground floor levels and will remain there in any foreseeable design circumstances. The problem is therefore a combination of security monitoring, so that any unauthorized lifting of manhole covers or access panels can be identified early enough to prevent subsequent actions or sabotage, and of siting, so that such access only leads into otherwise empty and secured spaces.

## Access monitoring and security systems

While on the subject of access via the services systems, the actual alarm, security and detection systems should also be considered. It should be remembered that, conventionally, the building services designer is responsible for the incorporation of such systems into the services design. Security, as with fire defence, is a specialized topic which at the terrorism level is becoming more and more technologically complex. Some building services design practices may not have sufficient expertise in this topic to carry out all the necessary specification and design work and it is obviously desirable that they consult the specialists. There are also similar practices with sufficient expertise to identify and analyze the problem, produce a solution and specify the requirements quite clearly to suitable equipment suppliers and installers. In either case, designers who are aware of security needs will readily discuss the problems with specialists, if only to keep abreast of the rapid developments in this field. A cautionary and possibly contentious note concerns the ability of the 'specialists'.

There are consultants who specialize in this field of activity and can produce a comprehensive design covering the complete range of security systems, either wholly independent of equipment manufacturers, or with minimal manufacturing facilities only for special equipment. At the other end of the scale are large manufacturing organizations which cover the whole range of systems, frequently with separate technical and sales organizations for each system. Each systems division offers a technical service but the co-ordination into a comprehensive whole by the controlling organization is sometimes lacking.

It is certainly difficult to achieve this co-ordination at times, whatever promises are initially made. If the client has not, via his architect, nominated a specialist professional security consultant, then the responsibility will lie with the building services designer. He will then have to make the decision on the correct direction in which to proceed, dependent on the client's brief, the complexity of the system and his own knowledge of the subject.

What is the best solution? I suggest careful selection of designers who have a broad-based knowledge of the possible problems with sufficient specialist skills to either provide, or obtain, the solutions.

At any particular level the designer must have the ability to decide where external specialist help is required and this includes the consultation with various police authorities.

It is not the purpose of this review to provide details of specific security systems but this is a suitable point to illustrate the range of security and associated systems for which the building services designer is normally responsible. They include:

Door, window and access point alarms

Perimeter fence monitoring

Closed circuit television, internal and external

Acoustic, laser and similar systems

Various fire defence systems

Electronically controlled access cards

Communications

Data control centres and software.

At the same time the designer must not contravene central or local government laws on safety or technical requirements, and this may create difficulties in high security buildings.

## Cutting off external supplies and standby plant requirements

Apart from the activities of terrorists or saboteurs within the building, or explosive attacks on its exterior, consideration must be given to possible external activities against the services feeding the building. This is one of the elements listed in Fig. 1, and Fig. 3 indicates the range of these services.

## Electricity

The most vulnerable service to the majority of buildings is almost certainly the electrical supply. Fig. 4 is a schematic of the various forms of electrical supply which may be encountered in practice both in the UK and elsewhere. The use of the various forms for different types of buildings is illustrated in the table which is associated with the figure. Some of the more complex options may be adopted independent of terrorist activity, because of the non-reliability of the external supply. The schematic is only a guide to the possibilities, in many cases the transformers would be part of the sub-station.

The schematic and its table are virtually self-explanatory. The simpler forms of supply lend themselves to damage or destruction without any risk to the terrorist or vandal who only has to obtain access to unprotected sub-stations.

In essence, the higher the security risk the more independent sources or non-interruptible standby supplies become necessary. When one designs the more sophisticated on-site generation systems, problems of space, noise, fuel storage and suitable maintenance become additional design features.

## Other fuels

With the exception of power stations, coal supplies are unlikely to be at risk for any of the buildings considered in this paper. Oil and gas may be at risk. Where gas is the primary fuel its protection should be treated in much the same way as described for electricity. Like electricity, site storage is minimal but unlike electricity the site production of gas is highly improbable. Therefore, where gas is used, some form of dual firing equipment should be employed with the secondary fuel being oil and the storage capacity being adequate for the estimated maximum gas outage period. In the UK gas tariffs for large installations do include an interruption clause which may, in theory, be for periods as long as 60 days, so that oil storage would in any case be required against this eventuality.

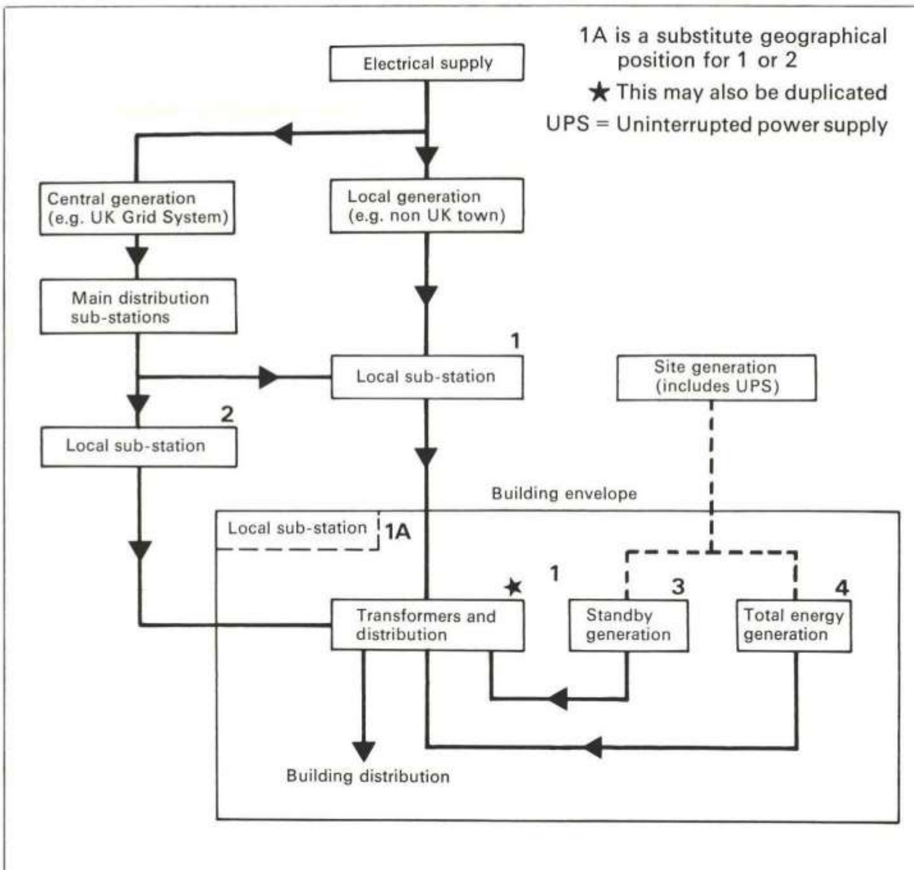
Where oil is the primary fuel it is less likely that there will be an alternative fuel. Where oil is supplied, either by pipeline or tanker, the storage capacity must in fact be suitable for full load operation for the estimated maximum period for which supplies are not available. On highly secure buildings a strike of tanker drivers may be just as critical as any terrorist activity.

In any case where fuel is being stored, the positioning of large storage tanks, where security is a priority, will frequently create building design problems at both the planning and detailed design stages. The tanks have to be secure from unauthorized access but accessible for checking leaks and fire fighting purposes.

## Communications

For security purposes the maintenance of communications external to the building is vital. The most accessible communications link for the terrorist is the telephone system. In the UK the Post Office have a monopoly on hard wired communication routes into a par-

**Fig. 4**
Possible electrical supply situations




*Supply options for various buildings*

| Type of building | Form of supply | | |
|---|---|---|---|
| Normal | 1 | 1+3 | |
| Special activity, e.g. computer, process | 1+3 | 1+2+3 | 4 |
| Secure, e.g. government, communications | 1+2+3 | 4 | 1+4 |

ticular site or building complex but elsewhere this may not be the case and there may be independent sources of incoming communications lines.

It is possible to cut such incomers both literally and metaphorically and to stop normal hard wired data transmission. This does not mean that this is a common event by reason of terrorism, vandalism, fire or flood, but it has occurred. It is therefore imperative to ensure that, where necessary, there is a duplicate incomer at a separate intake, served from a different exchange and that other forms of communications are also available which are not cable-dependent. Obviously telephone exchanges themselves are particularly vulnerable, a point clearly understood by the terrorists concerned with the incident previously mentioned.

### Water

The water supply to a building or even an area can be prone to attack and dislocation, as was illustrated by the example of a pipe line explosion in Wales a few years ago. Depending on location, multiple water inlets from separate sources are recommended. Apart from the necessity of maintaining the supply for normal security purposes, the needs of fire defence systems are extremely important in the context of security.

As with oil, storage facilities have to be provided of a suitable capacity to satisfy the needs of domestic hot water services, drinking water, HVAC, cooling towers and fire defence for a specified period of time. These facilities should be separated for the various services and stored in different areas of the building.

### Introduction of harmful substances

The elements so far cover damage or dislocation caused to building services by destructive means. It is also possible that the services themselves may be utilized by terrorists (or vandals) to create havoc within buildings, without any attempt at personnel infiltration. This refers to the fourth point of Fig. 1, the introduction of various harmful substances into air, water or fuel which is being supplied to the building. It is not intended to cover the detailed possibilities in this particular field of activity but certain necessary features of preventive design are outlined.

The suitable positioning of air intakes and extracts has already been mentioned to prevent unauthorized access of personnel. The adoption of the same design philosophy is equally necessary to prevent the introduction of substances into the air distribution system. The filtration equipment and monitoring of air quality may be extremely important on a high security building, even with the intakes designed to be inaccessible. This may be extended to include alarms and emergency actions in the event of contamination being detected. It should be remembered that filtration equipment is very selective in terms of the large range of possible noxious substances which may be introduced and that this form of security may be very costly.

The introduction of dangerous substances to the water supply is also one which may be accomplished from outside the building boundary. Unlike air it is not possible to remove the access points beyond reach. While this form of activity seems unlikely it should be referred to as one link in the security chain which may have to be considered. One basic solution is to take all water through secure primary storage tanks, which are monitored regularly or continuously for possible forms of contamination, and then pump the tested water from them to the main storage tanks.

The contamination of air or water will affect personnel directly. The contamination of fuel may bring the plant to a halt. The degree and form of contamination may affect certain fuel uses more than others. Here again such actions are unlikely but there is a potential security hazard now and possibly more so in the future.

**Fig. 5**
Some specific building services security problems

| Source of problem | Danger related to services | Solution |
|---|---|---|
| Danger from within | Electrical dislocation | Restricted access to electrical distribution centres |
| | Data passed out via waste systems | Filtration/monitoring of waste products (wet and dry) |
| Fire defence systems | Alarms out of action | Protect electrical distribution |
| | Water supply cut off | Duplicate sources and prevention of unauthorized access |
| | Water damage by vandals | Prevent unauthorized access |
| Lifts | Lifts disabled by vandals | Lighter and brighter aesthetic surroundings<br>Stronger car equipment |
| | Lifts damaged by terrorism | Prevent unauthorized access |
| | Unrestricted access to and from lifts | Individual or centralized control by card or key, programmed for security |
| Lighting | Cutting off electrical supply | Include maintained and emergency (self-powered) systems<br>Prevent unauthorized access |
| | Damage to luminaires by vandals | Select special luminaires<br>Integrate luminaires into design of building fabric<br>Remove from easy reach |
| Controls | Any service is vulnerable if its control system is inoperative | Secure electrical supplies<br>Duplicate wiring by separate routes<br>Restricted access control centre with duplicate/back up equipment remote<br>Security and checks on software |

### Specific problems

There are a number of specific problems which do not easily fit into the first four elements of Fig. 1 and are categorized as the fifth item. Fig. 5 tabulates several of these and outlines the dangers with proposed solutions. A more detailed approach is given below.

### Dangers from within

Restricting access to buildings and plant has been emphasized as being the best solution for safeguarding major systems. If, however, the staff includes personnel whose reliability is not absolute then problems can be caused from within, related to all aspects of security, one of which is the continuing operation of building services. There are specific areas of building services which are relevant in this context:—

(1)(a) The complete disruption of the electrical system at a key point and hence the possible interruption of security and communication systems

(b) Sabotage of air-conditioning and heating which would disrupt the building operation

(2) The less obvious but effective use of waste disposal systems to pass out data, thus avoiding the normal security procedures. This point is clearly related to security measures which may not be terrorist inspired.

The remedy to the first is very high security at the electrical distribution key point(s) and for the heating and ventilating plant rooms, while sophisticated filtering/monitoring arrangements will overcome the second.

This, as with several of the other points, sounds rather like the world of James Bond, but emphasizes the need for security screening to a suitable level, based on the risk. It also highlights the need, under certain conditions, to introduce special features into certain basic building service systems.

### Fire defence

In building services terms this covers a very wide range of systems in its own right. These are related to security of personnel and property and are part of the overall security system of the building. Like other security systems the integrity of fire defence systems needs to be assured and this is covered under normal design procedures. Where terrorism is considered as an additional fire hazard, it is essential that alarm systems, water storage and wet and dry risers be designed into the building to minimize possible damage from the terrorist activities which precede a fire.

If we consider the mundane but more frequent occurrence of vandalism and the fact that fire defence systems must always be operational, whether the building is occupied or not, then the access situation becomes crucial. Most buildings do not fall into the high security category of this overview. If they are not secure they may be entered by vandals (or thieves) and the vandals can cause more damage with water than they can by breaking whatever is breakable. Fire defence systems once installed are part of this risk as, to a lesser extent, are any other wet systems in the building. One can only pose a question in this respect. How far does education or discipline need to go to reduce vandalism, as an alternative to the provision of security systems and guards to prevent unauthorized access to unoccupied buildings? The answer cannot be given within the terms of reference of this paper as it is a complete subject in its own right.

### Lifts

Vertical transport has many advantages to the user. It may also suffer from the effects of vandalism, may be subject to the requirements of the security system and does offer terrorists an opportunity to create panic. For obvious reasons details of the last item will not be pursued, other than to say the saboteur must be denied access to the lift system. Some comments on the first two items are necessary.

Vandalism in lifts is regrettably well-known and occurs in rather specific types of building, such as local authority high-rise housing. It creates inconvenience and delay rather than, of itself, being costly to correct. Solutions are probably long-term. Brightening the decor and better lighting of the lobbies would help as would more sturdy control equipment in the lift cars and lobbies. This is no substitute for people respecting the environment in which they live and this must be an educational process. Manning the lobbies is not a suitable solution as this would need 24 hour shift working in the particular problem areas being discussed.

In security terms lifts may be used to provide access to particular floors and deny access to others. Keys or electronically accepted access cards are two methods by which selective access to and from lifts, or any other areas, may be achieved. The not uncommon 'Directors' Lift' in high-rise offices is a simple example of selective access. In addition to this, it is not difficult to override any lift controls from a control centre such that it may be stopped, started, sent to or held at a particular floor, with doors open or closed, in accordance with security requirements. The simplest example of this is the automatic arrangement which runs all lifts to the entrance lobby when there is a fire alarm.

### Lighting

This is mentioned for completeness in a review of building services and security. Suffice to say that buildings these days are commonly designed with normal, maintained and emergency lighting, even without high security risks. So long as this is done with special risk buildings and the electricity supply is maintained as indicated earlier, there is unlikely to be a problem.

Lighting which is installed in unprotected areas subject to vandalism must be carefully selected, integrated as far as possible into the building fabric and kept out of easy reach.

### Controls

Controls have a very wide meaning depending on the profession of the designer to whom the term is used. In normal use for building services it generally refers to the thermostatic controls and data centres relating to the HVAC system, but it does of course also cover lighting controls, fire defence, lifts and security systems, etc. Where there are complex systems, with or without sophisticated security requirements, the use of computer-based data centres and, now, microprocessors is an accepted design philosophy. A cautionary note should be sounded. Do not accept any system without independent professional advice. The manufacturers may not always be totally objective when offering their own systems for this activity.

It is common for data centres to be located in an area or space set aside for the purpose (Fig. 3) but access is often unrestricted. When high risk security is involved, the area itself becomes the key point for security operations. This area then becomes a segregated and secure space in its own right, normally within the building which is already protected. It is important to decide whether this area is common for security and plant surveillance, whether it is split physically with separate staff for each function or whether the monitoring equipment is centrally located with repeater and/or backup systems for specific purposes, e.g. fire or security on 24 hour manning.

In design terms, if the control centre equipment is to be a co-ordinated whole, the requirements must be considered at the brief and concept stages of the project and all the design disciplines need to provide input at these stages.

Apart from this co-ordination, the duplication of wiring systems run through different routes for all security-orientated systems is a technical requirement which affects several design disciplines.

The use of computerized data and control centres also requires an appreciation of what suitable software is available and what has to be written for a particular project. It should be made clear that the costs of any special software may frequently be very high and that the period for debugging the new programs may run well beyond the date of beneficial occupation. Security of the software packages themselves must also be taken into account.

### Costs

One can only generalize on this subject. Almost without exception any building, its services, facilities and security, are governed by cost. There is an analogy between absolute security and the item of absolute flexibility in many briefs for buildings. To achieve this total flexibility implies virtually infinite costs, and so it is with absolute security of services. In the design of services for security purposes the cost of the measures taken must be related to the probability of the risk events actually occurring.

The security brief for any building, as with any other section of the brief, must be clearly expressed at the concept stage by the client and refined into specific requirements by his professional design team so that the cost implications are clear from the outset. The cost effect of some of the suggested standby arrangements may be enormous on a large complex, e.g. electrical plant and storage volume. However, with the exception of particular items such as these, good security costs very little in relation to overall costs.

On very high security buildings it is essential that the designers co-ordinate their efforts to reduce the special cost element. Non-communication between designers will be infinitely more expensive than on the normal building.

### Conclusions

The conclusions which may be drawn from this review are best summarized as a list of the points which are important to the building services design engineer in terms of security planning. Many of the items apply equally well to the more conventional aspects of building services design. It should be emphasized that very few buildings are likely to require the ultimate in security precautions but the principles should be employed irrespective of the degree of sophistication.

(1) Adequate brief for the level of risk
(2) Co-ordinated and integrated design
(3) Suitable specialist input
(4) Cost criteria
(5) Restricted access
(6) Suitably distributed services for the risk
(7) Maintenance of essential services and standby arrangements
(8) Monitoring and centralized surveillance equipment
(9) Suitable communications–internally and externally
(10) Inclusion of full testing and commissioning procedures

**Fig. 6**
Principles for building services designed for security purposes

The summary of principles is listed in Fig. 6, and it should be noted that it includes a requirement for comprehensive testing of security systems.

In the context of security perhaps the wisest dictum is that of the Scout movement, 'Be Prepared'.

### Acknowledgements

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# Angel Court: Cladding

**Architects:**
**Fitzroy, Robinson & Partners**

Len Jones

### Introduction

The Angel Court development is situated close to the Stock Exchange and the Bank of England in the City of London. The prominent position of the site predetermined that a high standard of exterior finish was expected. This report attempts to illustrate the design implications involved from a structural viewpoint; we feel sure that an architect's report would be quite different in its concerns and priorities. The architects' concept was to have a glass and polished granite continuous face to the building rising to a height approaching 100 m.

Previous UK experience on the use of granite in high rise buildings was found to be remarkably limited. It was feasible to fix granite sheets of an appropriate thickness in the traditional manner, despite the increased wind, rain and temperature design ranges which would be encountered.

An early examination of the implications on the anticipated contract period suggested that by using conventional in situ applied granite, either this became the critical item and would dictate the overall contract time or a labour force of approximately 80 experienced fixers would be required. Such a large labour force was unlikely to be available. Neither course was attractive and, coupled with the daunting prospect of checking the installation of all the concealed fixings required for in situ work, the attention of the design team was directed to other possible techniques.

The initial concept of attaching the granite to a prefabricated support backing had the advantage of reducing the site work and the significant advantage of eliminating an external scaffold from the building. The next major step in the design was to reverse the construction process and, instead of applying the granite as a finish to the support backing, the reinforced concrete backing would be cast directly onto the back face of the granite.

The architect proposed to use a mahogany-coloured granite, imported from North Dakota, USA, its principal feature being the colour and consistency and its freedom from veining and streaking. Because of the highly reflective surface and the requirement to have a continuous flush face for the full height of the building, emphasis had to be placed on all aspects of dimensional control, movement and material performance.

### Design parameters

Whilst there is an abundance of technical literature available on precast concrete and a limited range of information on post casting applied finishes, we could not obtain reference to the use of granite and concrete as a composite unit.

Our principal doubt was whether the bond between the granite and the concrete could in fact be relied on in the design. In the opinion of the granite supplier, as well as ourselves, debonding of the interface due to shrinkage movement in the concrete was likely to take place relatively soon after casting. From mock-ups and the subsequent production units this debonding has been observed.

Because of uncertainty on this point in the early design stages we considered both the bonded and unbonded conditons. In the bonded case, shrinkage, temperature and creep would cause dimensional distortions in the composite unit; whilst in the unbonded case the granite would move independently of the concrete backing giving rise to movement at each of the mechanical fixings.

When the granite to concrete bond has broken, the interface becomes vulnerable to the possibility of water moving along the interface by capillary action. Repeated freezing and thawing actions could then cause a gradual breakdown of the granite at the fixings and a potential failure condition. We decided it was essential to design an effective sealing system for all the joints to the perimeter and between individual granite panels.

The thickness of the granite if designed in accordance with CP298 *Natural Stone Cladding* should be 40mm. As the code was concerned with in situ applied cladding it was felt that this had only limited relevance to the proposed system where a continuous backing would be formed. From considerations of working and handling the granite sheets, combined with the depth of penetration of the fixings, it was feasible to use the granite more as a veneer of 25mm minimum thickness. This approach was discussed and agreed with the District Surveyor.

This had obvious advantages in terms of the quantities of material that had to be shipped from America and the resulting costs.

We had thus reached the stage where the major parameters had been set:

(1) A combined granite/concrete precast unit would be adopted.

(2) No external scaffold would be needed.

(3) Work on site would be minimized.

(4) The granite would be treated as a veneer.

(5) Dimensional stability, due to the reflective effect of the finish would have to be maintained.

(6) High performance sealing of junctions would be needed.

### Structural stability
#### *Design loads*

The wind loads were assessed from *CP3 Chapter V* 1972, resulting in a maximum pressure of $\pm$ 2kN/m². This was subsequently reconfirmed by interpretation of wind tunnel tests carried out at Bristol University. These tests were originated to investigate environmental aspects of the development, coupled with verification of the service engineer's assumptions on intake and extract air positions. We obviously took advantage of the model to confirm all the structural loadings.

The window frames, which were designed for the same wind loading are supported by the cladding units and are restrained vertically and horizontally along their base, i.e. at the cill level of the spandrel unit. The top and side frame fixings provide lateral restraint but are free to move vertically.

#### *Material tests on granite*

Tests were carried out at Aston University to determine the mechanical and thermal properties of the granite. The results are tabulated below.

| | |
|---|---|
| Tensile strength | 4.07 N/mm² |
| Shear strength | 21.2 N/mm² |
| Modulus of elasticity | $4.75 \times 10^4$ N/mm² |
| Coefficient of thermal expansion | $8.55 \times 10^{-6}$ per °C |
| Thermal conductivity | 2.1 W/M °C |

#### *Thermal movements*

The Imperial Mahogany which is dark in colour and highly polished was expected to retain very high surface temperatures when exposed directly to ultra violet sunlight. In idealized still air, surface temperatures of the order of 85°C were estimated, but for design purposes, accounting for movements of air across the unit, it was reasonable to assume 60°C as the maximum surface temperature. A value of −8°C was taken in the cold range.

Two aspects of the temperature movements had to be considered: first, the overall movements of the unit within the structural frame and secondly the differential movements between the granite and the concrete. This would result in movement at the individual granite joint positions in the unbonded condition or warping in the fully bonded condition.

In the design it was assumed that the total unit was initially at a temperature of 20°C and that the external face of granite suddenly attained surface temperatures of 60°C in the hot range and of −8°C in the cold range. As the maximum differential movement is time-dependent, i.e. the steady state temperature gradient is not the critical condition, this instantaneous raising of the surface temperature is a somewhat extreme boundary condition. Fig. 1 indicates temperature distributions across the composite section with respect to time for a sudden increase of surface temperature.

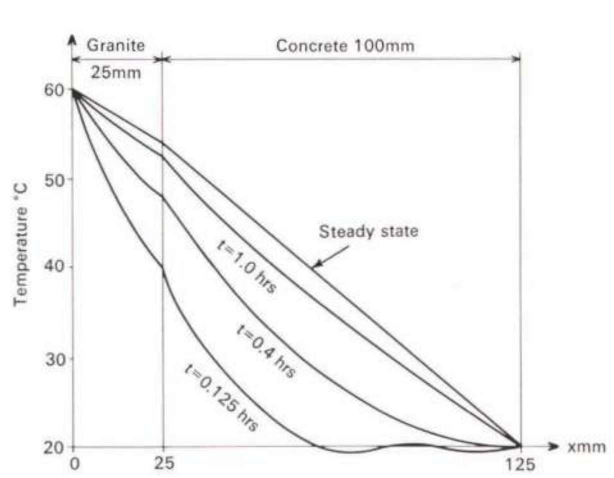


**Fig. 1**
Temperature profiles at various times

### ***Shrinkage and moisture and creep movement***

Whilst the shrinkage of the concrete could not be prevented some of the influencing factors could be controlled within practical limits.

The rate of shrinkage is a function of the standard of curing, whilst the total shrinkage movement is a function of the concrete composition, its thickness, reinforcement quantity and atmospheric humidity.

In assessing the joint movements, a mix design had to be assumed which could reasonably be reproduced by the unit supplier. Certain of these aspects were incorporated in the specification such as:

(1) Limiting w/c ratio (Maximum)

(2) Limiting a/c ratio (Minimum)

(3) Aggregates with low drying shrinkage

(4) Fine aggregate rounded to Zone 1 or 2 grading

(5) Curing standards.

The remaining movements due to moisture changes were not considered significant.

As the units were not primarily structurally loadbearing, the creep movements were negligible.

### **Granite and granite fixings**

The approach normally adopted on in situ applied granite is to independently support each panel using a corbel plate near the lower edge of the panel and ties near the top edge. We were concerned that if we used this system the anticipated movements between the granite and concrete backing would be restrained as the cavity which exists on in situ work would not be present.

With individual granite panels of sizes up to 1.7m x 1.35m, we decided that by using 6mm diameter stainless steel dowels a better system was obtainable, and less than total reliance would need to be placed on the effectiveness of every fixing. By inclining the dowels they would individually act as both corbels and ties, and by introducing rubber washers at the interface we would improve the movement characteristics. The system is shown on Fig. 2. On each panel the inclination of a small percentage of dowels was reversed to prevent complete detachment of the granite.

### **Joint details and sealants**

The primary function of the joint filler was to seal the edges of the granite panels against water penetration. The performance of the seal was to be effective with all combinations of movement.

The materials considered were polysulphide mastic and silicone rubber. In general the performance, life and readily available colour range of the silicone rubber were better but it suffered from the disadvantages of being more difficult to apply and a general lack of experience of its use in similar situations.

We opted to specify a polysulphide mastic with a reversible movement capability of ±12.5%.

The effective life of polysulphides could not be guaranteed but as the work was to be carried out under controlled conditions at the precast manufacturers works it was generally expected that 20 to 25 years was probable. As the deterioration of the polysulphide would be very gradual and routine maintenance inspections would not be entirely reliable, it was decided to incorporate a second back up system of sealants. This system was the preformed neoprene seals shown on Figs. 3 and 4 which could be inserted and deformed to produce a compression seal.

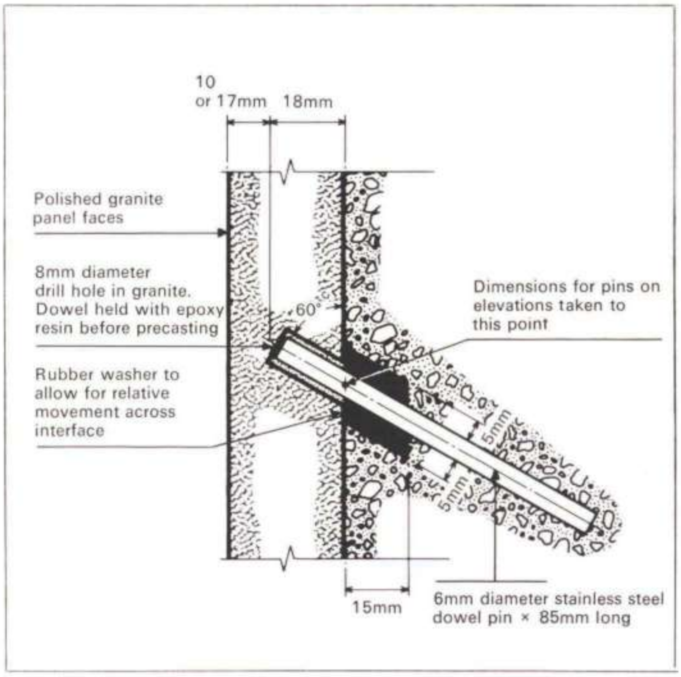


**Fig. 2**
Dowel pin detail

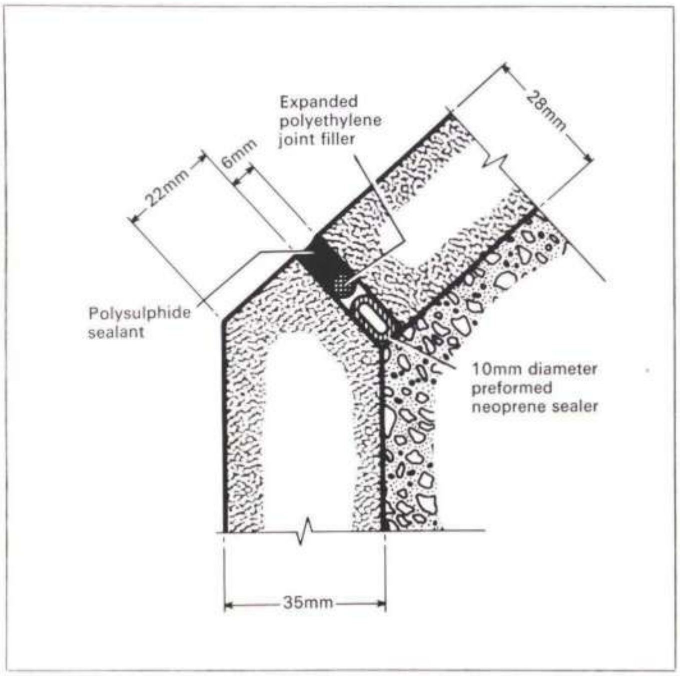


**Fig. 3**
Granite/granite joint

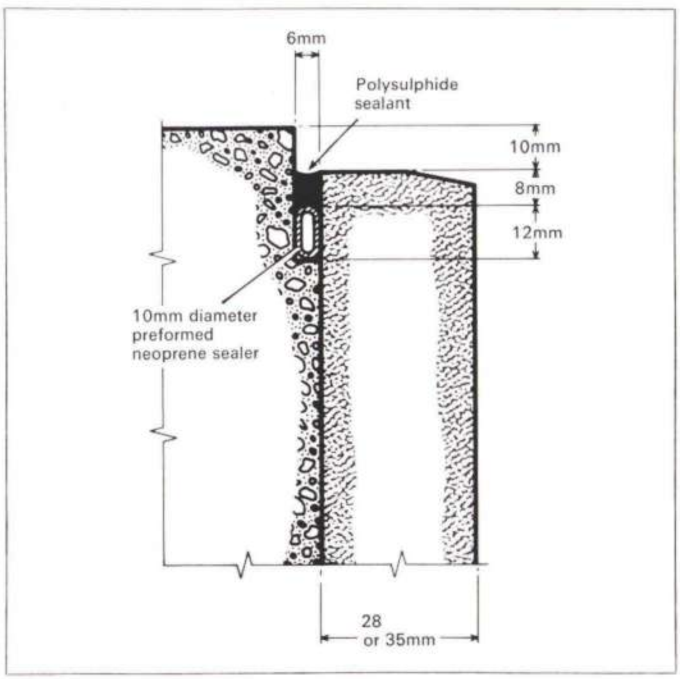


**Fig. 4**
Granite/concrete joint

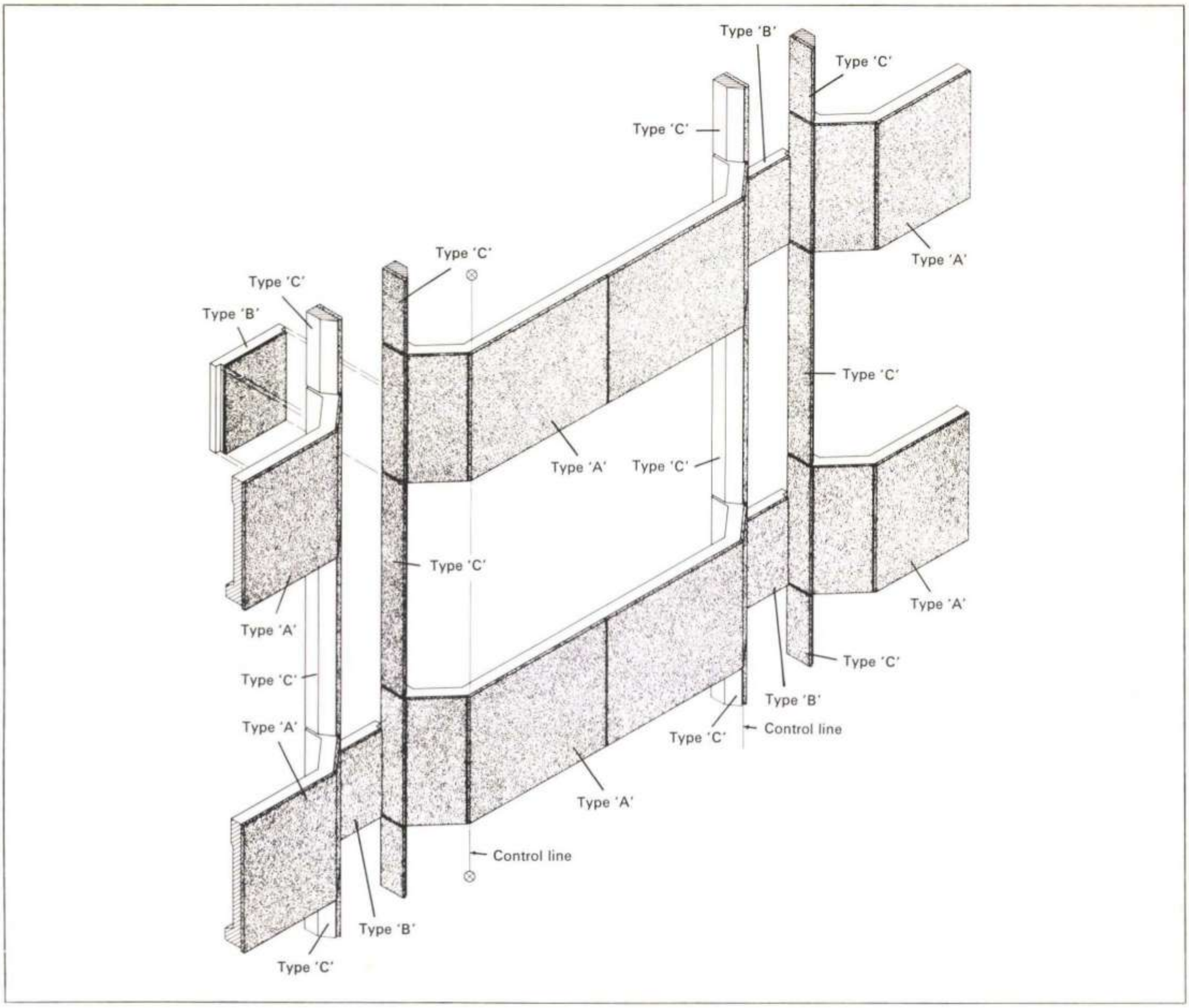


**Fig. 5**
Isometric projection
of precast cladding units


**Fig. 6**
North podium and tower at June 1977
(Photo: Sir Robert McAlpine & Sons Ltd.)

### Production drawings

It was decided to arrange, by negotiation with Bannocks of Birmingham, the supply of the granite. We were thus able to secure early ordering of the blocks from the quarries, even in advance of the appointment of the main contractor. Bannocks were subsequently appointed as a nominated supplier to Ibstock Precast Ltd., as the nominated sub-contractor for the supply only of the cladding. The erection of the units was kept within Sir Robert McAlpine's main contract.

Although there were a number of options on the extent of the detailed information that Fitzroy Robinson and Partners and ourselves would supply to the contractors, the adopted procedure was for Arups to produce a complete set of fully detailed manufacturing drawings from which both Bannocks and Ibstocks could work without the need for either of the companies to produce further drawings.

**Fig. 7**
North elevation of tower at August 1978 (Photo: Sir Robert McAlpine & Sons Ltd)

**Fig. 8**
Throgmorton Street elevation at June 1977 (Photo: Sir Robert McAlpine & Sons Ltd)

The drawings incorporated all the architect's requirements, McAlpine's agreed lifting and temporary fixings, the window manufacturer's cast in fixings, and our own requirements. A further set of drawings for the assembly of the units was produced to cover the McAlpine work on site.

Whilst this approach gave us a significant increase in the draughting requirements on the project, almost 100 drawings being finally needed including key elevations, it did mean that one master set of drawings existed which contained all the essential information. We avoided the fairly lengthy and sometimes involved procedures which would have been needed if the detailed drawings had been produced by the sub-contractors for the architect's and our approval.

### Supply and manufacture

The granite was quarried in Dakota and roughly shaped into blocks 10 x 6 x 5 feet. It was then shipped via the Canadian great lakes and the St. Lawrence River onwards to Liverpool. From there it was generally taken to Matlock, Derbyshire, to be sliced into panels of the correct thickness; some blocks, however, we were led to believe, even found their way to Italy for this initial work.

The sawn slabs were then taken to Birmingham where Bannocks carried out all the final preparation of the individual panels. The work included all the grading, cutting to size, grinding, polishing and drilling for fixings. The finished panels were then transported to Ibstock Precast Ltd.'s main works at Clifton near Rugby for the production of the units, including all joint sealing.

The schedules of concrete/granite units show that over 2,000 were manufactured, the majority with a high degree of repetition. Even so, all the moulds used by Ibstock were constructed in timber, one of the prime reasons being that all the finally exposed faces were of granite and surface deterioration of the mould would not result.

### Schedules of units

| | | | |
|---|---|---|---|
| Standard | Type A | Spandrels | 412 |
| | Type A | Derivatives | 154 |
| Standard | Type B | Column infills | 190 |
| | Type B | Derivatives | 75 |
| Standard | Type C | Jambs | 902 |
| | Type C | Derivatives | 140 |
| Standard | Type D | Corner infills | 112 |
| | Type D | Derivatives | 42 |
| | Total number of units | | 2027 |

### Acknowledgement

Although this was a large contract by almost any standards the care, thought and co-operation imparted and experienced by all members of the design team, the contractors and manufacturing teams made this an almost trouble-free contract.

It is particularly regrettable, therefore, that since the completion of the contract, due to the national recession in the UK construction market, Ibstock Precast have decided to withdraw from tendering for specialist work of the type they so successfully carried out at Angel Court.

### Credits

*Client:*
The Clothworkers' Company

*Architect:*
Fitzroy Robinson & Partners

*Quantity surveyors:*
Gardiner & Theobald

*Main contractor:*
Sir Robert McAlpine & Sons Ltd.

*Precast manufacturer:*
Ibstock Precast Ltd.

*Granite supplier:*
Bannocks of Birmingham

# Buxton Opera House

**Restoration:**
**Arup Associates**

Buxton has an opera house, built in 1903, designed by the eminent architect, Frank Matcham. The auditorium is beautifully figured and painted by Italian craftsmen and seats about 950 people; its stage is bigger than that of Sadler's Wells Theatre and the dressing rooms, wing space and flying facilities are adequate for any major opera production. The fact that it is a Grade 2 Listed Building and has to be preserved is a national recognition of its great quality. Between 1903 and 1939 the theatre was continually presenting the major artists of the day—Sir Frank Benson, Charles Doran, Martin Harvey, Bransby Williams, George Robey, Albert Chevalier, Pellisier and so forth. The London Old Vic Company ran Summer Theatre Festivals just prior to the War with Robert Morley, Emlyn Williams, Anthony Quayle, Alec Guinness, Andre Morell, Andrew Cruickshank and Diana Wynyard.

The Opera House is a unique stone-clad building, which forms an essential part in the fabric of the Spa. It is part of a remarkable entertainment complex comprising a concert hall designed by Robert Rippon Duke in 1876, a pavilion designed by Edward Milner in 1871 and a playhouse dating from 1892, linked by bars, dining rooms and conservatory. The surrounding gardens with their ponds, streams and pagodas were designed by Edward Milner and Sir Joseph Paxton who had previously co-operated on the Crystal Palace Gardens in London.

The current proposals for the Opera Festival were originated by Malcolm Fraser, Anthony Hose, James Tomlinson and David Rigby who will be amongst the directors of Buxton Opera Festival Ltd. Their proposals were stated as follows:

'The renovation of the Opera House is to be linked specifically to the presentation of an annual Festival of Opera of international standard. The Festival is to operate on the same artistic level as such major British Festivals as Edinburgh, Aldeburgh and Bath. We are aiming at both a local and a visiting audience.'

The opening performance will be Donizetti's *Lucia di Lammermoor* on 30 July 1979.

1

**Fig. 1**
The auditorium before restoration shows the intimate nature of the Opera House
(Photo: Arup Associates)

**Fig. 2**
The auditorium as a cinema
(Photo: Arup Associates)

**Fig. 3**
Buxton Opera House built in 1903. The Pavilion Winter Garden and open gardens are to the left
(Photo: Arup Associates)

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**Fig. 4**
The auditorium ceiling
before restoration
(Photo: Arup Associates)

**Fig. 5**
Auditorium ceiling. Plaster mouldings
being redecorated
(Photo: F. Jewell-Harrison)

**Fig. 6**
The proscenium and boxes
seen from the dress circle
(Photo: Arup Associates)

**Fig. 7**
The dress circle stripped out
and undergoing restoration
(Photo: F. Jewell-Harrison)

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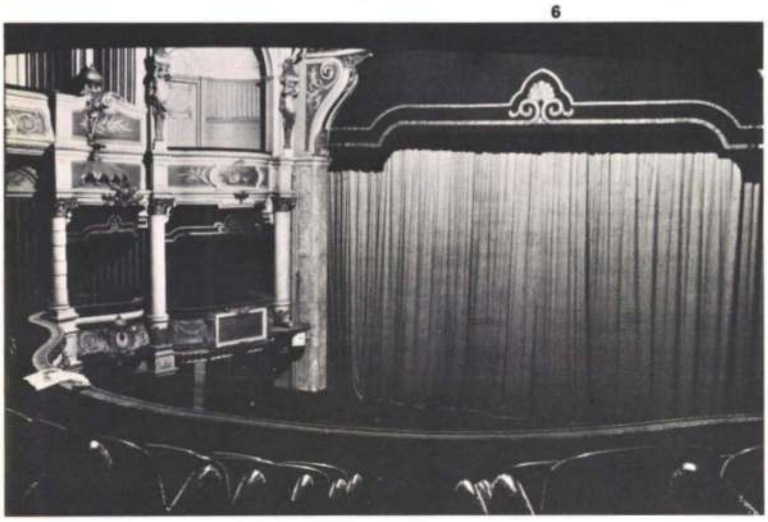

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**Fig. 8**
Main foyer staircase before restoration
(Photo: Arup Associates)

**Fig. 9**
The main foyer and staircase during restoration
(Photo: F. Jewell-Harrison)

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