THE ARUP JOURNAL

SEPTEMBER 1970

THE ARUP JOURNAL

Vol. 5 No. 3 September 1970
Published by
Ove Arup & Partners Consulting Engineers
Arup Associates Architects and Engineers
13 Fitzroy Street, London, W1P 6BQ

Editor: Peter Hoggett
Art Editor: Desmond Wyeth MSIA
Editorial Assistant: David Brown

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Front cover: I warehouse, St.Katharine dock, seen from the new Dock House (Photo: Henk Snoek)

Back cover: Cameo of Thomas Telford (Reproduced with the permission of Mrs. N. S. Telford)

An engineer looks at architecture

Ove Arup

This paper was given at Leicester University Arts Festival on 11 February, 1969

Introduction

The title for my talk is *An engineer looks at architecture*. The engineer is me, of course, and the only reason for bringing him into the title is to indicate that I am *only* an engineer, and cannot claim to be an expert on architectural theory, so what I say about architecture may not meet with the approval of the pundits. But I have had a great deal of experience in helping architects to design buildings and other structures, and I have tried my hand at the latter myself, and it was always the intention that the result should rank as architecture, preferably good architecture. Unfortunately that doesn't mean that I can give you any satisfactory or generally accepted definition of the term, and I doubt whether anybody else can. I can only give you my own point of view.

A building can be a piece of architecture. But of another building, some might say that it isn't architecture. In other words, buildings are architecture only if they fulfil certain conditions. Which conditions?—general confusion.

Architecture can also mean a discipline, like philosophy, medicine, law, building science, sociology or Art—with a capital A. In fact architecture was once considered the mother of arts, a valid art form like painting, poetry or drama, and many attempts have been made, and are still being made, to produce a theory of architecture which will enable us to distinguish architecture from mere building, good from bad architecture, and which will be a guide to the *creation* of architecture.

These attempts to distil the volatile essence of architecture from the many tangible proofs of its existence encompass such acknowledged landmarks as Vitruvius's *De Architectura*, and Norberg-Schulz's latter day *Intentions in architecture*, many less systematic but more impassioned pleadings by actual and would-be architectural practitioners, and the weekly outpourings of esoteric critics in architectural journals whose claim to fame rests on their being incomprehensible, or so it appears to a mere engineer.

What all these have in common is:

1 that they are all different, if that isn't too Irish

2 that they deal almost exclusively with aesthetic considerations of order, balance, space, form, light and shade, textures—in short, the visual organization of the material, and are less concerned with function or economy

3 that although they may be a very valuable aid to the understanding of different architectural viewpoints, they are too vague to be a guide to architectural creation. What I mean is, you cannot look up in a book on architecture where to draw the next line on a drawing board, as the engineer can if he has forgotten the formula for analysing the strength of a beam. Books and drawings may enable an architect to produce a building in the Gothic or even Miesian style, but that is not nowadays considered an acceptable substitute for true and original architecture—whatever that may mean.

No, the architect nowadays is definitely on his own, in an ocean of complete permissiveness and an almost infinite choice of means. It can be a relief, therefore, if his choice is limited by impossible site conditions and a shortage of materials or technical resources.

A corollary, or in any case in keeping with this chaotic state of affairs, is the fact that architecture cannot be said to have progressed from old times till now. Few would argue that modern architecture is a marked improvement on the architecture of earlier epochs, that our great architects are greater than their ancient confrères. Perhaps such a comparison is meaningless. But look at the contrast in engineering. Here there is steady progress in the achievement of engineering aims, and a very much greater agreement about what is good and bad engineering.

What is the reason for this difference, and how can it be maintained in view of the fact that there is no clear border line between engineering and architectural structures. Whether you call a bridge architecture or engineering is optional—if we attach the normal vague meaning to these words it is both. And that in fact applies to all man-made structures—they are both structures—i.e. engineering structures and pieces of architecture, in different proportions, if you like.

The question arises, can one separate the structural from the architectural part? Can a structure be a bad engineering structure and good architecture or vice versa? That is a very important question, which I will return to later, when I have dealt with my first query.

The reason for the difference between the architectural and engineering 'climate', so to speak, is very complex. It is partly a matter of terminology, partly a matter of historical accident, and the consequent training of architects and engineers, and mostly a matter of what is commonly supposed to be the difference in content or context—architecture being concerned with producing works of art; engineering with utility structures.

Structural requirements

If we continue to use the word structure in its widest sense, in which it can mean a building, a silo, a bridge, dam or road, in fact anything built by man which stays put—for a time at least—so as to avoid 'static hardware' or some such concoction—we have seen that to call some of them architecture and others engineering structures is rather arbitrary.

We demand of all of them that they should function well, last well, look well and cost little, but if we survey the whole field of possible structures, the emphasis placed on these four demands differs widely.

They must all fulfil their particular function, for that is the reason for building them. But the functions vary, from those which are easily defined but difficult to fulfil, for instance to build a bridge over a gorge, to others which are easy enough to fulfil if only we could manage to define them. A teaching hospital, involving several authorities and a large number of doctors all with their different and often conflicting demands, and which moreover may very likely become obsolete before it is finished, owing to changing demands, belongs to the latter category.

They must also all last well—that is, they must be stable and able to withstand wear and tear by natural forces or imposed loads. But this may be a simple or a very complicated matter —the latter a characteristic of daring engineering structure.

They should also all look well—they create our man-made environment which is of concern to us all. But again the emphasis varies widely—between, for instance, a retaining wall and a cathedral.

And finally they should all cost as little as possible, but again, the need for economy varies. Economy is a matter of devising a sensible way of building the structure, and even the richest client doesn't want to spend more than necessary.

Roughly speaking engineering structures are those which have an easily defined and undisputed function and which present structural problems of some intricacy, whereas those where aesthetic and functional problems dominate are classed as architecture.

Or we could say that the first are those where a civil engineer is in charge and responsible for the design, whereas the second are designed by architects.

The two professions

These two ways of classifying structure lead to more or less the same result, and correspond to the difference in training and background of the two professions. At least this was the case before the drastic changes brought about by the modern movement in architecture and the technological advance since then. Engineers built bridges, railways, harbours, dams, tunnels, factories—in fact all the utility structures called into being by the industrial revolution, whereas architects were concerned exclusively with 'fine' buildings, with Building as an art, an art which could trace its origins back to antiquity. Ordinary houses were at that time, and are still to some extent, the province of builders. Architecture was confined to mansions or important public buildings and these were designed by architects on quite different principles. The architect then had a client who knew, or thought he knew, what he wanted, and a well-established tradition of buildings based on a few materials and supported by craftsmen who knew their job and were proud of it. He could therefore concentrate on creating architectural masterpieces according to the prevailing rules of the game. In fact the highlights of architectural achievement were felt to lie in the past rather than in the future. Engineers looked the other way.

It was in part the great engineering achievements in the last century which triggered off a change in architectural philosophy. The Bauhaus, the whole Modern Movement, accepted the new structural materials and the technological advance with enthusiasm and dreamt about building a new and better world. It partly misfired, for enthusiasm for the visual manifestations of technology was not enough to effect a radical change in architectural thinking—the Modern Movement in many cases amounted to no more than a new style. But one radical change in outlook took place which changed the way we look at architecture today. Architects acquired a social conscience, they were no longer content to cater for the whims of rich clients, they dreamt of a better world where technology could make everybody share in the good life. The client was therefore society as a whole, and the people actually using the buildings. They felt it as a duty to help in formulating the brief, a task which grew more and more complicated, and which demanded the expertise of architects and engineers. The field of architecture was widened to include housing, schools, hospitals—and gradually all structures which clutter up our environment, for their architectural quality concerns us all.

At the same time engineering technology also invaded the whole field of building and construction. In fact the realization of architects' dreams depended on engineering knowledge. The whole way of building has changed—it will soon all be engineering.

The need for co-operation

This demands collaboration of architects and engineers, a topic which has been discussed *ad nauseam* these last 30 or 40 years.

To begin with, the civil engineering consultants, the heirs of Telford and Brunel, were reluctant to enter the field of ordinary building—there wasn't much engineering in it, it was fiddly, unremunerative work and they were certainly not prepared to act as mere assistants to architects—they didn't think much of them anyhow.

The architects, on their side, naturally wanted to retain complete control over the design—you have to, when you are producing a masterpiece, you cannot tolerate interference from people who know nothing about the finer points of architectural composition—or for that matter, from those who do. But a new proletariat in the form of structural engineers grew up, somewhat more narrowly based than civil engineers, but with a deeper knowledge of engineering structure, and willing to offer their help under the architect's direction—the normal process of specialization. And many other kinds of engineers and experts are coming into the picture. Buildings nowadays are full of services and gadgets which account for a greater and greater part of the total cost.

All these changes have brought about a pretty chaotic state of affairs. People, old habits, old ways of thinking, do not change so rapidly. There are powerful personal interests at stake. New ways of building demand larger jobs to be tackled by larger organizations, different financial arrangements, collaboration of different departments not in the habit of speaking to each other, in fact a complete reorganization of the whole building industry. At the moment every possible combination of new and old methods and attitudes is in evidence.

It would take far too long to sort out this complicated situation now—and I am just as lost as anybody else anyhow. But one thing is obvious—what we build is always a whole, an entity—a building, a precinct, a town with roads, etc.—and all these entities interact and influence each other. If you split the design of these entities amongst a number of specialist designers each acting more or less independently, you won't get an entity, but a hotch-potch. There must be co-ordination, integration, a proper assessment of priorities based on the true interest of the community.

At the moment we are far from achieving this integration. We are just about drowning in specialization. It is perhaps the biggest problem we have to face.

It means much more than the much discussed collaboration of the design team. Design decisions are taken by all sorts of authorities, financiers, etc., outside the design team proper. It means, for instance, that the Ministry of Transport must collaborate with local planning authorities, that city architects must work hand in hand with city engineers and extend the other hand to those who frame our bye-laws and building regulations and to government on a local and national level. And it means further that the gap between design and execution which is such an ingrained feature of the building industry must be bridged somehow. For the designer must be familiar with the means of execution and their cost. It is a fallacy to believe that more accountancy and cost planning will in themselves make building more efficient. These obviously have their use but chopping something off a design by lowering standards does not create anything. For that you must go back to the drawing board and make a better design. To get the right materials in the right place is what gets building costs down. Design is creative accountancy.

The future

To use our resources efficiently there are two kinds of question we have to make up our minds about: what to build, and how to build, needs and means, more or less! Of course they are intertwined in various ways, for what we decide to build depends largely on what we can afford to build, and this depends again on the means available or employed. So that those who decide what to build must be advised by those who know how to build. But I think this distinction can help us to throw some light on the role of engineers and architects in building.

How to build will more and more become a matter of engineering, it will at least have to employ engineering methods and will need an attitude of mind mostly found among engineers. That the designers must be thoroughly familiar with the means of construction and their cost has already been mentioned—as I have said so often, designing means indicating a sensible way of building.

The engineer, however, is used to working to a definite brief. What he has to achieve is fairly easily defined—to span a river, to store grain, etc. He is dealing with *the art of directing the Great Sources of Power in Nature for the use and convenience of man*—but he does not give so much thought to whether it will in fact benefit man. He will see to it that the thing functions well, last well and costs little, provided somebody tells him what the function is going to be. And the 'looks well' part is not always his strong suit. He has both feet on the ground, not given to dabbling in airy-fairy aesthetic notions, or so it is generally assumed.

The modern architect, on the other hand, is endeavouring to design an environment for people to live in and work in and enjoy—a very different and very much more complex business—for we humans enjoy so many different things.

A house is a machine to live in—Corbusier's dictum, which has loomed large in the public image of modern architects, has perhaps not always been rightly interpreted, as Joe Chamberlin pointed out in his lecture to the RIBA in 1969. The machine part is largely the engineer's business—but *to live in* is not a simple matter. Man lives not by bread alone. Man needs life and company as well as calm and privacy, he needs the sun and air and green plants and birds singing as well as the excitement of the metropolis, he needs work and play and relaxation—he wants to be able to drive his car anywhere—but he doesn't want his peace disturbed by other cars. And his needs tend to grow faster than our means to satisfy them. Man is quite a problem.

Architects have long been grappling with this problem. It cannot be solved by computers or technology—it is a question of sensitivity, of understanding the deeper needs of human nature.

Amongst these experts the architects should play a decisive role, for it is their task to ensure the artistic quality, the character, the visual harmony of our environment.

For me, then, everything we build is architecture—different kinds of architecture, even when the structural problems dominate. An engineering structure is not a good structure if it is not also a pleasant thing to look at. And a piece of architecture is not perfect if the structure is second-rate, if the roof leaks and the heating fails.

Pre-stressed concrete congress, Prague, June 1970

David Dowrick

The 6th congress of the Federation Internationale de la Précontrainte was held in Prague and attracted 2300 delegates from at least 49 countries including Kuwait and Monaco but excluding China. About 80 British delegates attended including Arupians Povl Ahm, Duncan Michael, Ken Shaw, Keith Ranawake and myself. The Arup contingent was swelled by the presence of our South African colleagues, Ron Heydenrych and Ian Scott who brought their wives, as did Povl Ahm and Keith Ranawake. Most of the British delegation stayed at the International Hotel, a vast pile of a place, massive bureaucratic architecture with primitive plumbing and non-automatic lifts—all very Russian. Try the Park Hotel if you go to Prague.

In order to accommodate the vast numbers the congress was held mainly in the Julius Fucik Hall with some smaller meetings in the Technical University. The congress timetable was very crowded, taking five and a half days from 9 a.m. until 6 p.m. and included lectures, reports, technical papers and some visits to local building sites. It is hoped that the following account will give a reasonable idea of the wide ranging nature of the material presented.

Lectures

B. Gerwick (USA) opened the proceedings with a fascinating if sometimes rather futuristic lecture on floating or submerged structures in prestressed concrete, his subjects ranging from ships to undersea oil tanks and undersea nuclear power stations. With the latter two types of project the problems of controlling cracking must become onerous indeed on we poor designers, not to mention the contractors.

Y. Guyon (France) discussed composite structures of steel and prestressed concrete, a combination of materials with which I was unfamiliar. He drew a number of examples from France and Belgium, and claimed that some advantages could be found for the system on certain projects. The idea apparently is to combine the constructional advantages of steelwork and the stress advantages of prestressed concrete. When questioned on economics, M. Guyon hedged his bets even more than we usually do in engineering. Unfortunately M. Guyon spoke far too quickly and most of his message was lost.

R. Baus lectured on fatigue and breakdown of prestressed concrete and presented so much detail that he completely failed to communicate to such a huge audience, especially via translators. The only reason I mention his lecture is to make the point that speaking to a large international audience requires a special approach and anyone intending to do so should consult a few past audience members first!

The rules are:

1 speak slowly

2 use simple language as far as possible for your theme

3 any slides must be extremely clear and varied with very few words which must be printed as large as possible

4 avoid using complex equations

International recommendations for the design and construction of concrete structures

Something of a landmark in engineering affairs was witnessed with the presentation of these recommendations jointly agreed by the CEB and FIP. This new document is a step forward on the 1966 draft recommendations, incorporating many amendments and additions to the original text.

One important facet of the document is the adoption of SI units and it struck me as still rather strange to see Newtons rather than Kgf in an international document. It is significant, however, that concrete stresses are quoted in N/cm^2 rather than N/mm^2, allowing an easier comparison with the European Kgf/cm^2. International co-operation is certainly not at its best regarding what one might have hoped would be the simple, quantitative issue of units. It is apparent that confusion will reign over this subject for many years to come.

Another point of comparison with intended British practice is the terminology for design limit states. The CEB/FIP document prefers 'ultimate limit state' to 'limit state of local damage'. I personally prefer the CEB/FIP terms, the word 'ultimate' being widely used internationally, and 'serviceability' being more meaningful than 'local damage'. The proposed British unified concrete code will take enough getting used to without such new terms as well as all the other changes.

An interesting inclusion is the recommendations for dimensioning of punching shear reinforcement in flat slabs.

Commission reports

Several FIP Commissions presented reports on subjects ranging from fire resistance to prefabrication. Each commission has an international membership and does much to draw experience from widely differing sources and increase the chances of general acceptance of the recommendations.

On durability

This to me was one of the most significant reports as it brought an encouraging rebuttal to those doubting Thomases who keep predicting that all our prestressed structures will have short lives due to insidious dangers such as corrosion of tendons, creep, relaxation, etc. A world-wide survey has been carried out which examined the state of 200,000 prestressed structures and has shown an extremely low proportion of cases causing concern. Most of the damage was repairable and the majority of reported accidents date from the early days of prestressing techniques and many of the causes are unlikely to be repeated owing to the advantages of experience. Certain types of high alumina cement are banned, and the grouting of cable ducts is fully discussed and recommendations made.

Report on seismic structures

This report consists mainly of design recommendations prepared since the last congress and should be read in conjunction with the existing FIP/CEB document *Practical recommendations for the design and construction of prestressed concrete structures*. There are some useful comments on ductility and the design of columns and connections.

The report on prefabrication

This was discussed at some length and mentioned the stressing together of precast units, in particular Coventry Cathedral, Glasgow Airport, and the Sydney Opera House in relation to the use of epoxy glued joints. Of interest were the splicing of long piles (over 30 m (100 ft.) long), and a new system of tolerance control developed in East Germany.

Member group reports

Various countries presented group reports on recent major works under the three headings of Bridges, Buildings and Other Structures. The British report on bridges was delivered by D. Lee of Maunsell & Partners, and included the Western Avenue extension and the Calder River bridge, but, I thought, was not very well illustrated. In fact, it was striking how much more interesting the German and Italian bridge reports were than those of other countries. J. Sutherland presented the British report on buildings and, despite apologising for a lack of material, gave an outstanding talk which incidentally mentioned several Arup jobs. Ron Heydenrych presented the South African group report on buildings which included mention of our Standard Bank and Carlton Centre projects in Johannesburg. His description of the latter follows this article.

Individual technical contributions

This FIP Congress saw the innovation of technical contributions by individuals as distinct from group and commission reports. These were divided into English, German and French language sessions (without translation services) which were held concurrently in the Technical University. Thus these sessions were on a pleasantly human scale in terms of both lecture hall and audience size, compared with the impersonality of the vast Congress Hall.

Despite the shortness of time allowed for the presentation of these papers, they generally proved interesting, covering a wide range of subjects such as ground anchors and the prestressing of cylindrical wine reservoirs with heated cables. One of the more valuable papers dealt with a prestressed folded plate hangar by S. Firnkas (USA). This roof has a free span of 77 m (252 ft.) which is believed by its designers to be the longest free span folded plate ever constructed. It is certainly the longest I have heard of. The vertical structural depth of the folded plates is 4.27 m (14 ft.) Of interest is the means of dealing with the considerable horizontal and vertical movements in such a large roof, due to prestressing and gravity loads, shrinkage, creep and temperature effects. The whole roof sites on four spherical bearings situated on flexible corner columns. Full data on predicted and final cambers was given.

Three of the papers in the technical contributions sessions were about Arup jobs. Duncan Michael presented one on the suspended frame of the Standard Bank, Johannesburg, and I presented two papers, one on the Sydney Opera House roof and the other on the new foundations of York Minster. It was gratifying to hear Mr. Burr Bennett (USA) make very complimentary remarks about all three of these projects during his summary of the technical contributions to the full congress on the final day. As no Arup paper on the York Minster works has been in print before, we are reproducing the Prague paper on page 6 of this issue.

The exhibition

A most impressive display of technical expertise was given in the Congress Exhibition on behalf of 29 organizations from France, Germany, Italy, Switzerland, the UK and Czechoslavakia. Designers, contractors and suppliers of prestressing steels and stressing systems were represented.

It was most interesting to observe how the German stressing systems are generally promoted by the bigger contractors, in contrast to the British systems which are supply (and fix) only. There would almost certainly be more prestressing done in this country if there was a similar technical commitment to prestressing amongst our contractors. It is no accident that PSC and CCL are being forced into more and more site operations.

Of interest to Arupians was the stand of the

British Consultants Bureau. Roger Kemp is to be congratulated for his organization of the display of our work which included Gateshead Viaduct, Sydney Opera House, Standard Bank, Glasgow Airport and Lusaka Aircraft Hangers among other jobs.

Technical visits

Arrangements had been made with the Czechs for visits to various building sites around Prague, most involving prestressed concrete. Some of us chose a trip which took us to a warehouse and a bridge. The former was a very large storehouse for motor vehicle spare parts just out of Prague. The superstructure was entirely precast, including columns, prestressed trusses and roof slabs, while the walls were clad with *Siporex* slabs made locally under licence.

Although the roof trusses incorporated some nice design ideas, their construction left much to be desired, the members varying wildly in thickness and the reinforcement cage often bereft of any cover. One can only presume that the present political troubles have affected the morale and incentive of the men producing these goods.

We then visited the Nusle bridge, a major construction of several large spans across a deep valley in Prague. The bridge was of orthodox style and one felt that a lot more could have been made of the opportunities offered by the site to create a striking modern bridge in a city which once knew how to build beautifully. One point of technical interest, however, was the use of its cross-section which was of the single box girder type. The deck is to be used for road traffic while underground trains will run inside the box.

Social events

Thanks are due to the FIP committee who managed to arrange an evening of free entertainment for delegates who could choose between the State Opera at the National Theatre, the Czech Philharmonic at the Smetana Concert Hall and the Symphony Orchestra of Czech Radio at the Dvorak Hall. Those of us who went to hear the Philharmonic playing Dvorak voted it a superb performance.

Another evening we went to see the Magic Lantern which has to be seen to be believed—an extremely imaginative combination of films, puppets and live actors. Then on Friday night all delegates and wives attended a slap-up reception at the beautiful Cernin Palace (of Maseryk suicide fame). Oldrich Cernik was our official host but didn't appear for reasons which have since become clear. As an ally of Dubcek he has been obliged to relinquish his post.

Prague was a beautiful venue for an international congress, but also a sad one. The people speak surprisingly openly about their political troubles, and no-one likes the intervention of a foreign power in the management of their affairs. Ken, Duncan, and I had the pleasure of visiting the most hospitable families of two of our London colleagues and of helping, we hope, in some way to span the gap that divides them.

Conclusion

Although the very poor translation service was disappointing and diminished what we could get out of the Congress, there nevertheless was much that was stimulating, technically and otherwise. We all have many more friends in the industry, and the proceedings, when published, will be a useful document.

Finally I would suggest that, as well as the papers we presented, other Arup work was worth recording at the Congress in some way. James Sutherland actually had difficulty getting enough material for his summary of British prestressed concrete developments. An example of valuable experience which has not been properly published is John Nutt's account of grouting difficulties on Sydney Opera House roof. Who else is sitting on knowledge worth sharing?

The Carlton Centre

Ron Heydenrych

The Carlton Centre, which is situated in the business district of Johannesburg, is one of the largest central city complexes in Africa. The development comprises a 50-storey office tower, a 600-bed hotel and 4.05 hectares (10 acres) of shopping. Parking facilities are provided for 2,500 cars. In order to maintain open areas at street level, the majority of the shopping, parking and servicing facilities are situated in a seven-level basement. The basement extends over an area of 165 m (540 ft.) by 138 m (450 ft.) with the general level of the lowest basement varying between 23.5 m (77 ft.) and 29.5 m (97 ft.) below the surrounding streets. The variation in depth occurs because of a 6.1 m (20 ft.) cross-slope on the surrounding streets.

The site geology comprises residual soils from deeply weathered strata of various igneous and metamorphic rocks. A feature of the geology is the great variation in the consistency of the materials. The greatest design hazard, however, was the extreme fissuring of the parent rock. Generally, the natural water table was about 12 m (40 ft.) below the surface.

Studies of the economic viability of the project indicated that the earliest possible start on construction was vital. Consequently, it was decided to proceed with a preliminary contract aimed at excavating and stabilizing the basement. During the 12 months' contract period allowed for this work, the detailed design and documentation on the main contract was completed.

Two problems existed in ensuring the safety of the excavation; firstly, some form of strutting was needed to control the deterioration of the earth face and, secondly, a system of props was required to control the overall movement of the sides of the excavation.

Before any work was carried out on the site, the water table was lowered by means of a ring of well points surrounding the site.

In order to control the deterioration of the earth face and to allow for a simple propping procedure, 1.1 m (3 ft. 6 in.) diameter cast-in situ piles were installed at approximately 2.3 m (7 ft. 6 in.) centres along the perimeter of the site. These piles were, in general, carried down below the lowest basement level. Later in the contract, gunite arches were formed between the piles to complete a continuous permanent retaining wall.

The bending strength of the piles allowed bulk excavation to proceed without further propping to a depth of excavation against the piles of 10.7 m (35 ft.). This depth was the maximum clear height to which the piles could act as cantilever bulkheads.

The 29.5 m (97 ft.) depth and extent of the excavation made conventional inclined or flying shores impractical, but an essential requirement in all central city developments is the careful control of ground movements. Normal elastic shortening and temperature variations on any bracing structure of this size give rise to appreciable movements and provision must be made to absorb these movements.

In addition, the problems associated with constructing a building around a maze of supporting props are very real. The design, therefore, aimed at an open and relatively simple bracing structure, but nevertheless having facilities for incorporating sophisticated control devices.

Because of the fall of the ground, 6.1 m (20 ft.), the piles of two sides required propping at two intermediate levels, but on the other two sides a single prop at approximately ⅔ height was satisfactory.

The bracing structure took the form of a temporary rectangular prestressed concrete grid. While the principal structural behaviour of the grid was as a compression ring it was necessary to ensure full stability with unbalanced loading conditions and, for this reason, the additional struts across the site were added.

Fig. 1
The excavation for the Carlton Centre basement
(Photo: A. A. Gordon, Johannesburg)

The grid was cast on to grid piles and installed when the excavated level was about 9 m (30 ft.) below ground.

After completion of the grid, precast props were erected against the perimeter piles on the two deeper sides, while the bracing on the other two sides was achieved by casting the grid directly against the piles. Before proceeding with further excavation, the grid was prestressed and external forces were applied through jacks placed between the props and the grid.

The bracing grid represents an unusual usage of prestressed concrete in that the majority of the axial load was applied by external jacking forces. The grid was analysed under the anticipated loading conditions from earth pressure together with unbalanced loading applied over short lengths of the perimeter, in order to check the sensitivity of the design to abnormal conditions.

The stress conditions in the grid were complex and longitudinal cables were added in order to control peak stresses in the members and to increase the ultimate load strength of the grid. Prestressing was also applied transversely in order to resist the bending from vertical loads as well as to increase the factor of safety against beam shear failure. The grid was designed to be used as part of the construction access roadway to the site and was also used for the majority of storage areas and site offices. An important advantage of the design was the easy access for maintenance and adjustment of the jacks.

The grid was prestressed using 26 mm (1 in.) Dyckerhoff rods and, in general, unbonded cables were used to permit easy removal prior to demolition of the grid.

The design horizontal earth loads on the grids were of the order of 4300 kN/m^2 (40 tons/ft.^{2}). The axial loads in the members were very large and gave rise to elastic deformations of approximately 100 mm (4 in.) across the site. Similar movements of the order of approximately 50 mm (2 in.) were anticipated from temperature effects. In order to control movements of the supported earth face, 400 kN (400 ton) Freyssinet flat jacks were built into the seatings of the inclined precast props against the grid. Similarly, jacks were placed against each pile on the other two sides of the site, and, by placing a number of jacks in series, it was possible to allow for a maximum movement of 150 mm (6 in.).

Fig. 2
Artist's impression of the completed Carlton Centre

Hydraulic oil was used as the pressure medium in the jacks. The forces in the jacks together with the change in thermal conditions of the grid were monitored regularly and the jacks adjusted to suit changing conditions. The jacks were loaded to ⅔ of the full design load and increased to nearly full design load in those areas where the ground movements approached certain danger marks.

Stability of the basement in its final condition was provided by the permanent floor slabs of the building. As these slabs were constructed, the load in the grid was adjusted to transfer the loads to the permanent structure. After completion of the slabs, the grid was demolished with the help of explosives.

Prestressed concrete was chosen for this temporary grid in preference to structural steel for a number of reasons. Although demolition would have been easier with steel, prestressed concrete was markedly more economic, and in addition, the forming of the connections in steel with the very large loads involved would have created difficult engineering problems.

Although the grid represented a major engineering structure, its total cost, including demolition, was no more than 1½% of the total cost of the complex, and the saving in contract time afforded by the use of this temporary prestressed structure fully justified this cost.

Skidmore, Owings & Merrill and Rhodes-Harrison, Hoffe & Partners are the architects and the contractors are Murray & Roberts Holdings Ltd.

The use of prestressing in foundation strengthening at York Minster

David Dowrick

Summary

York Minster is one of England's great mediaeval cathedrals. It is about 158 m (515 ft.) long and 75 m (245 ft.) wide across the transepts. Its present superstructure was built between 1225 AD and 1472 AD, and it stands on the site of former Roman buildings which were followed by a series of Saxon and Norman churches. At the time of writing the Minster was undergoing extensive restoration to a total expected cost of about £2 m. A large part of this expenditure is due to underpinning and strengthening of the existing foundations, the inadequacy of which had caused substantial deformation of the superstructure. The basic trouble was that the area of the foundations was too small for the stratum of clay immediately underlying the building which had lead to excessive differential settlements and rotations.

There were three principal zones of damage:

1 the central tower area
2 the western towers
3 the east end

At the east end, the whole of the east wall was slowly but surely rotating outwards due to yielding of the clay under the toe of the footing. The structural solution was to replace the existing foundation with a new concrete one twice the size, but, as no prestressing was used, this will not be further discussed in the present paper.

In the central tower area, however, the greatest danger was not from continuing foundation movement. The cracking of the superstructure had been caused by differential settlement of the tower with respect to the adjacent structure, but little further settlement was likely. Thus although some superstructural control of the cracks was necessary, the cracking was important largely in that it provoked a study of the foundations.

After a good deal of investigation, including extensive exploratory excavation, it was discovered that the effective area of the footings was such that the factor of safety against shear failure in the clay was possibly as low as 1.3. Also the masonry below floor level was found to be badly deformed in bending and cracked in shear. The main problem was to strengthen the foundations of the four main piers supporting the central tower. A number of different strengthening schemes was studied, the one finally chosen having prestressing as its essential element.

The problem of the western towers was similar to that for the central tower foundations, and required a similar if simpler solution to that used for the central tower. Only the latter will be considered in the following discussion.

The central tower foundation scheme

The great weight of the central tower is carried mostly by the four main piers, each of which carries a vertical load of about 38,500 kN (3,850 tons) at floor level, while the smaller columns adjacent carry about 3,000 kN (300 tons) each. Exploratory excavations revealed that the columns were built on the remnants of the walls of the previous Romanesque-style Minster. These

Norman walls (c. 1070 AD) were in turn supported by strip footings, the tops of which were about 2 m (6 ft. 6 in.) below the present floor. The Norman footings were either 4 m (12.5 ft.) or 6.5 m (21 ft.) wide, and were about 2 m (6 ft. 6 in.) thick, apparently having been well founded on the virgin clay or on the very deeply founded Roman footings. Each of the four corners of the tower was treated separately but similarly, so it will be convenient in the following discussion to refer mainly to the NW pier. It was desired to make the effective foundation area about twice what it had been as well as to stabilize the cracked areas. As the main pier load was extremely high, and as the masonry was in a rather delicate condition, it was necessary to find a method of strengthening which would incur the least disturbance to the existing footings.

After studying various strengthening schemes, it was finally decided to encase the Norman footings in concrete, incorporating the NW pier and the adjacent nave and transept columns on one huge footing about 14.5 m (48 ft.) square. This would utilize all the cracked masonry footings as well as providing some completely new areas of footing in concrete. The total effect would be to spread the load from the columns over a much larger area of clay. The new average bearing pressure on the clay would be about 290 kN/m² (2.7 tonsf./ft.²) improving the safety factor against shear failure in the clay from about 1.3 to 2.5.

The new concrete and the ancient masonry were unified by prestressing them together. There are four layers of main prestressing rods, two layers in each orthogonal direction passing right under the main pier, thus providing the tensile reinforcement necessary in the bottom of this large spread footing. This main part of the new foundation is 2.1 m (7 ft.) deep with an average compression due to the prestress of only 0.86 N/mm² (125 lbf./in.²). The resulting stress regime can be considered as one of 'partial prestressing'.

In order to provide prestressing ducts through the foundations, it was necessary to drill 75 mm (3 in.) diameter holes through the ancient masonry. The following is a simplified summary of the construction procedure:

1 the first pour of concrete was cast including steel duct-tubes on the drilling side

2 the upper concrete collar was cast and the whole masonry footing was pressure-grouted in an attempt to fill any voids and cracks

As the new areas of concrete footing initially supported only their own weight, it was necessary to induce artificially their full load capacity. For this purpose 'compression pads' were cast below the main foundations. After horizontal prestressing of the whole foundation, the flat-jacks between the compression pads and the main foundation will be inflated, thereby vertically stressing the new foundation-bearing area; this second post-stressing action transfers the desired amount of load from the existing masonry to the new footings. (At the time of writing this process had not been carried out.)

Materials and techniques

Prestressing tendons

Because the whole strengthening work clearly had to be a once-for-all conservation operation with a life expectancy of hundreds of years, it was decided to make the prestressing hardware of stainless steel. The rods were made from a high-strength alloy manufactured by Firth-Vickers Stainless Steels Ltd. and designated FV520B. It was a precipitation-hardened stainless steel which had been overaged in a temperature of 550°C (1022°F) for two hours; a guaranteed minimum 0.1% proof stress of 800 N/mm² (52 tonsf./in.²) was agreed for this project. The ultimate strength averaged over 920 N/mm² (60 tonsf./in.²). These properties were achieved for the 32 mm (1¼ in.) rods used on site. The rods were continuously threaded which maximized their bond characteristics.

An initial prestress of 540 N/mm² (35 tonsf./in.²) was used. Prestress losses proved negligible on bars which were restressed several weeks after installation which is probably not surprising considering the comparatively low levels of stress in the rods in the foundation. Nevertheless it would appear that the relaxation properties of this stainless steel were favourable for prestressing purposes, but unfortunately no laboratory figures were available for relaxation at any stress levels.

Couplers

The maximum length of rod that could be readily manufactured was 5.5 m (18 ft.), and in certain places on site it was necessary to use shorter rods because of limited clearance for insertion into the ducts. Purpose-made couplers were used to join the rod, tapers being provided to facilitate threading the coupled rods through the masonry which was subject to partial blockage with chips loosened by the drilling. The couplers were 44 mm (1¾ in.) diameter and were made of the same steel as the rods.

Anchorages

The rods were anchored with a nut and an anchorage plate which were made of FV520B and AISI 304 stainless steel respectively. The rods and nuts had UNS threads, and the nut was the heavy duty size. All the above stainless steel items were fabricated by Frederick Mountford (Birmingham) Ltd.

Jacking

The rods were stressed using a Macalloy Mark 10 jack with a modified attachment to suit the UNS cut thread. The jack was very reliable and simple to use.

Laboratory tests

An independent authority was commissioned to carry out strength tests and tolerance

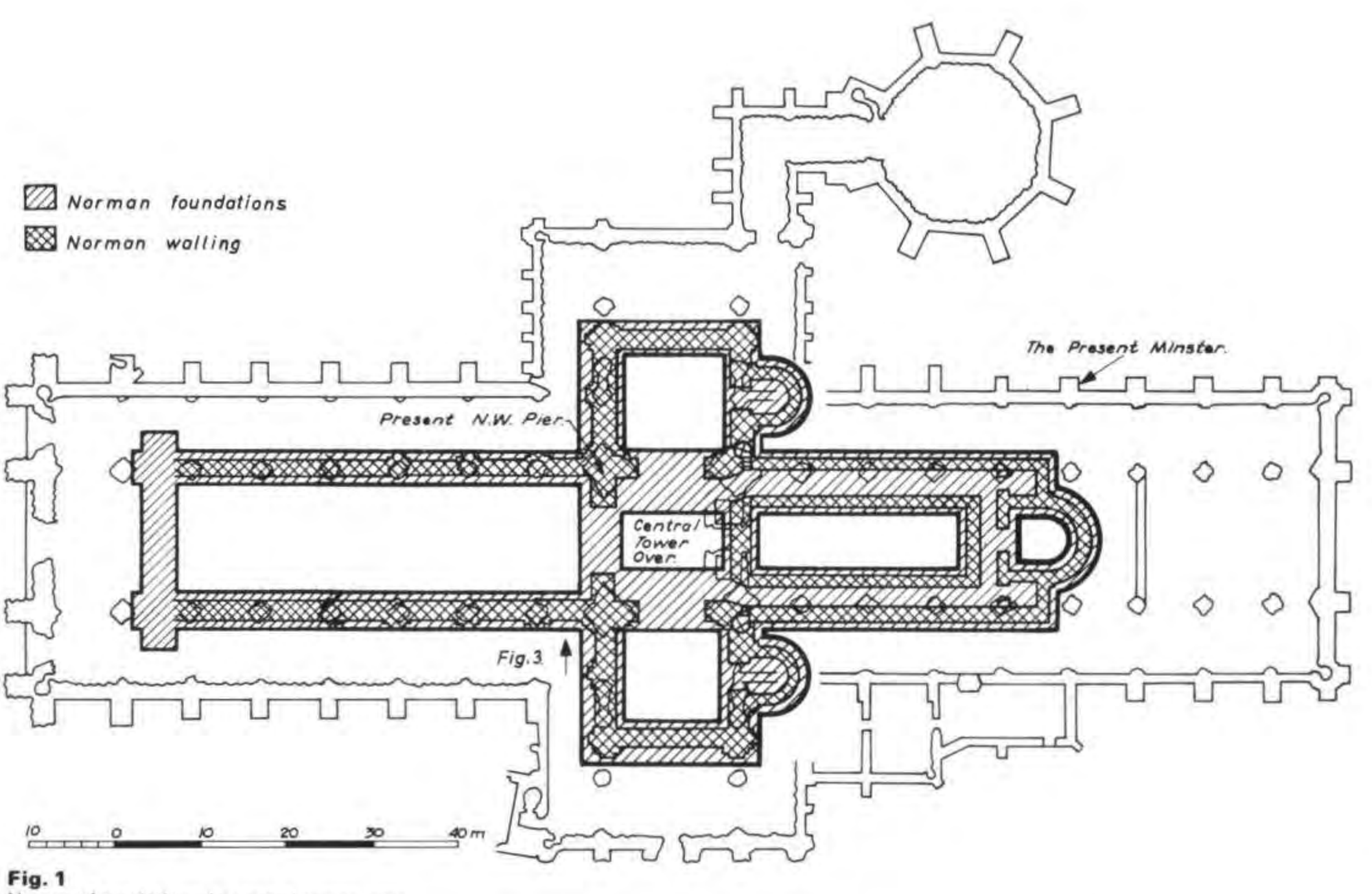
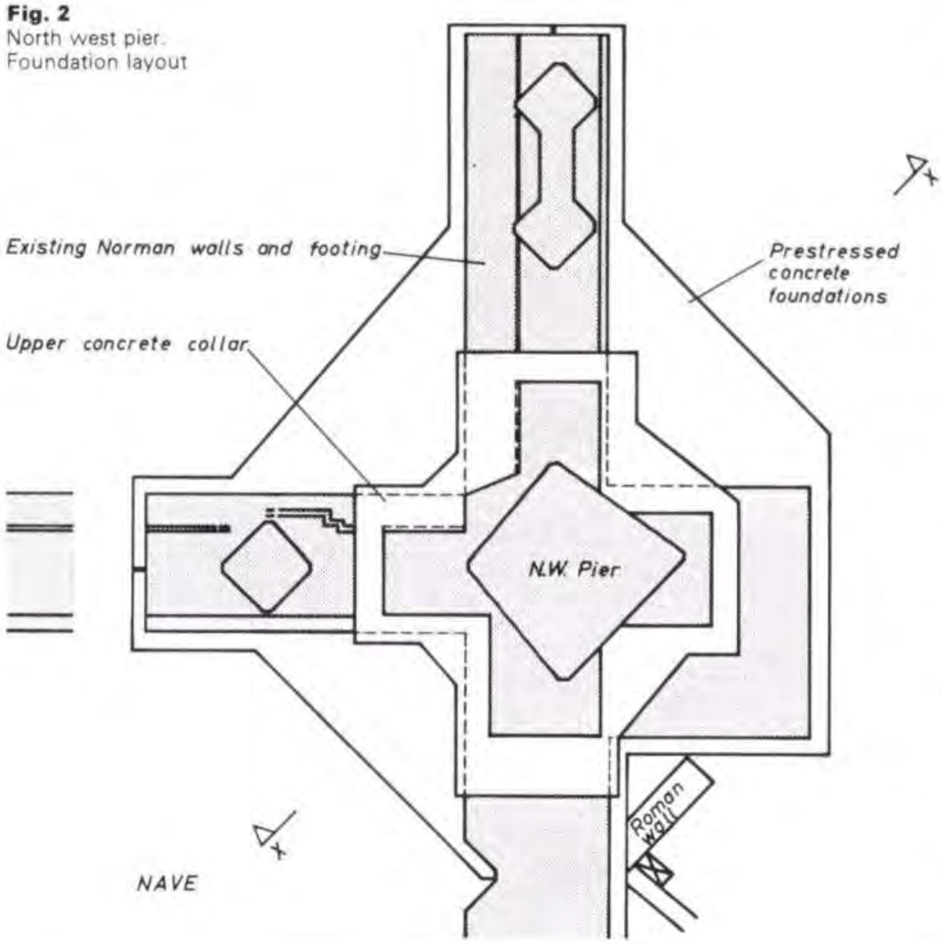


Fig. 1
Norman foundations in central tower area

Fig. 2
North west pier.
Foundation layout



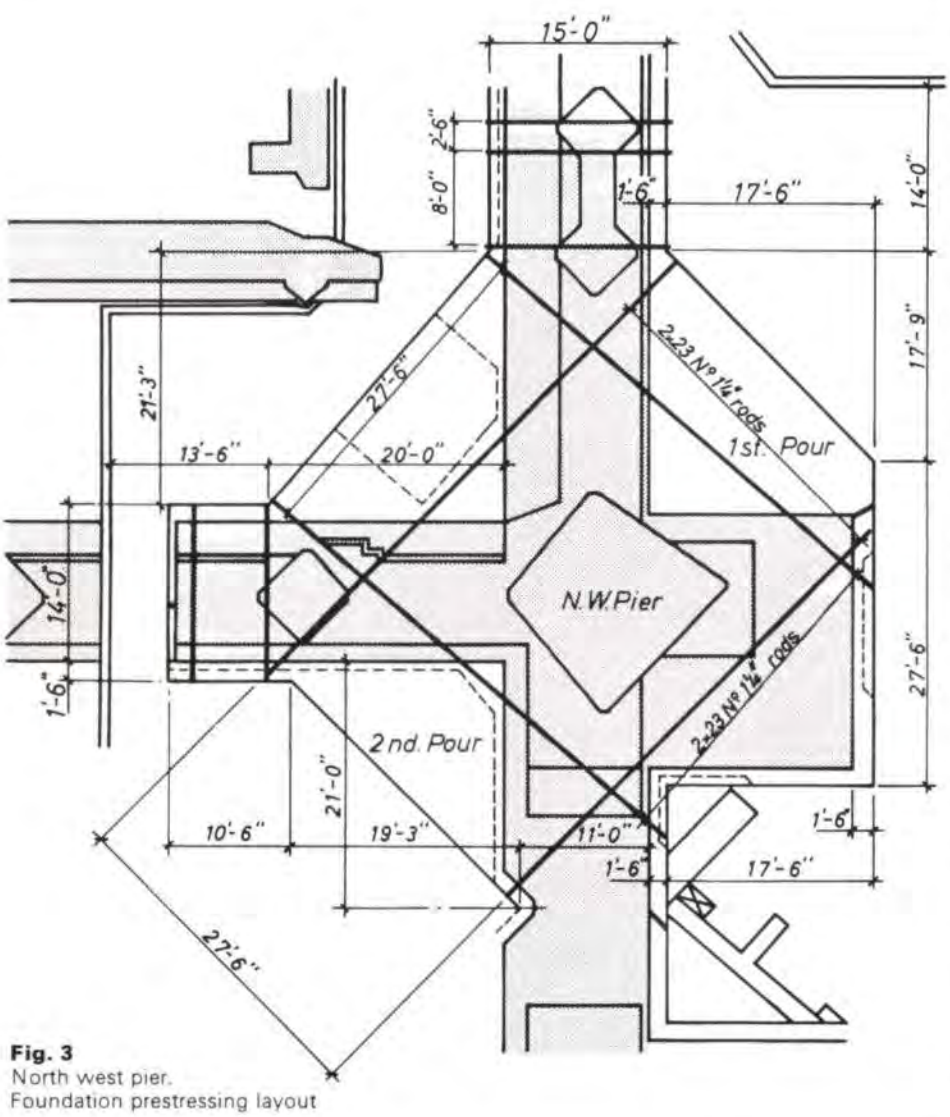


Fig. 3
North west pier.
Foundation prestressing layout

inspections of samples of all components throughout the production period. Some initial tests were done on the strength of the assembled rods, couplers and anchorages to ensure that the strength of the individual items was fully realized in the assembly.

Drilling

Right from the outset it was obvious that the success of the whole foundation proposal depended on the drilling operation. The drill shots varied in length from 6.4 m (21 ft.) to 16 m (50 ft.) through 11th century masonry of doubtful quality. Although the contractor had had wide experience in rock drilling, no one had any knowledge of the problems of accurate drilling through this type of material. The whole problem centred on obtaining the required drilling accuracy, which was determined by the alternate layers of rods being only 230 mm (9 in.) apart vertically.

An accuracy of 25 mm (1 in.) off line in the above length of shot is easily achieved when drilling in a homogeneous medium such as solid rock or good concrete, but this is not so in ancient masonry. Some preliminary drill shots carried out before concreting had begun were encouraging, indicating accuracies of the order of 25 mm (1 in.) in a length of 6 or 7 m (19 ft. 6 in. or 23 ft.). Also, during work already completed on the superstructure, 46 drill shots about 20 m (67 ft.) long had been put through the walls of the 15th century central tower. These had indicated that sufficient accuracy could be obtained provided all voids in the masonry were well grouted.

The drilling in the superstructure was carried out with a rotary-percussive tungsten-tipped pneumatic drill rigidly mounted on a special cradle. Unfortunately, when work really started on the foundation drilling, this drill proved to be too inaccurate, at least under the NW pier. The two chief factors contributing to this were firstly, the poor mortar in the 11th century masonry due to a very low lime content, and secondly, the presence of a grid of large oak baulks used by the Norman masons to reinforce their footings. Both these factors detracted from the homogeneity of the masonry. The attempt to solidify the masonry by preliminary grouting had not been as beneficial as desired.

After many rather dismal weeks of attempts with rotary-percussion drilling, the first really consistent success was achieved using a diamond-studded coring drill, with only rotary and no percussive action. While this system was being used to keep the job going further experimentation was carried out with a different rotary-percussive technique, and eventually a vole hammer drill proved to have about the same accuracy as the coring machine in this material. This was a most important development as the overall speed of the vole hammer was three or four times that of the corer and its overall operational cost was corresponding much less.

All the drilling equipment was supplied by Holman Brothers Ltd. and the very latest innovations in techniques were applied. At the time of going to press there was still much drilling to be done and further improvements may still be effected.

Acknowledgements

The above works were executed for the Dean and Chapter of York Minster by the following:

Architect: B. M. Feilden, Norwich.

Consulting Engineers: Ove Arup & Partners.

Contractor: Shepherd Construction Ltd. York.

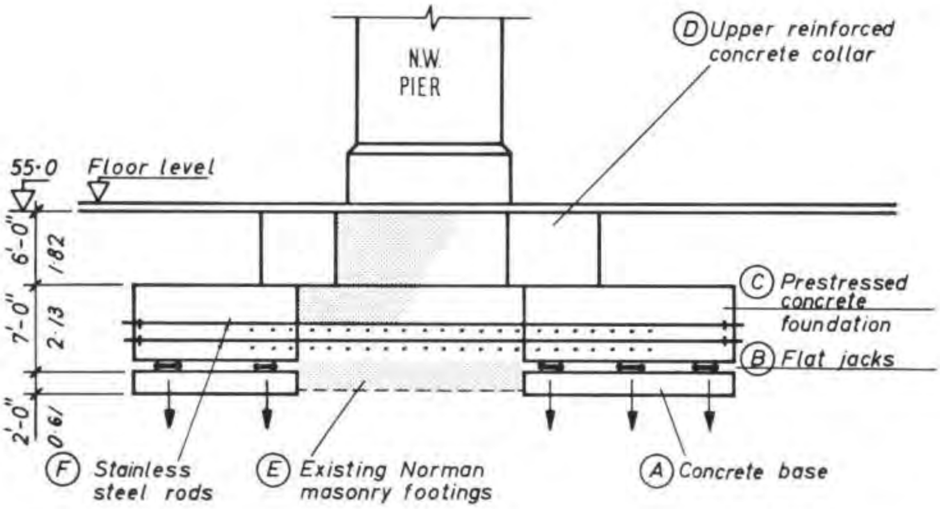


Fig. 4
Section through completed foundation

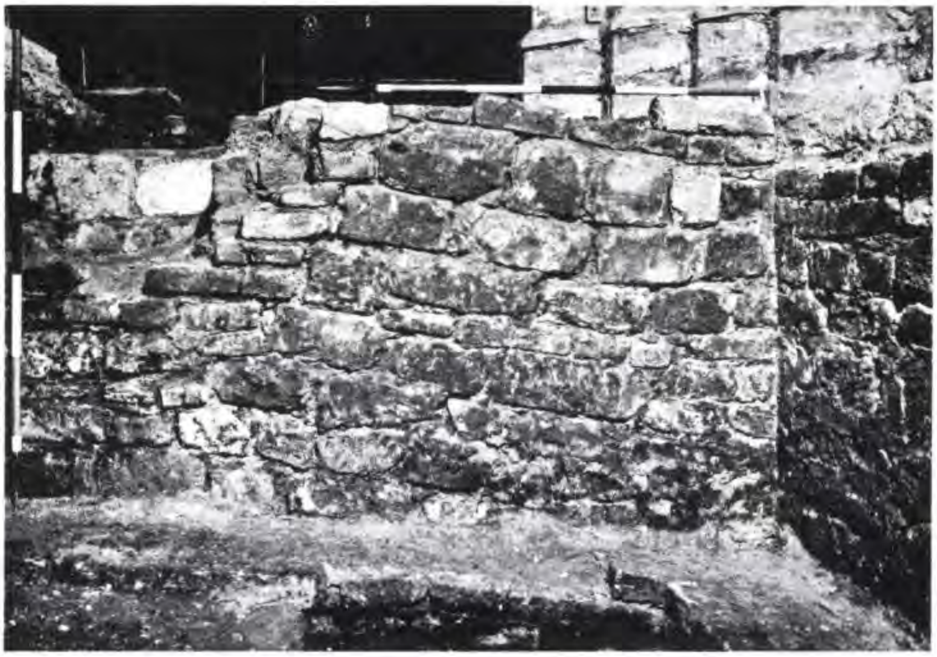


Fig. 5
Distortions in Norman walls below present floor level due to column on the right (Photo: by courtesy of the Royal Commission on Historic Monuments)

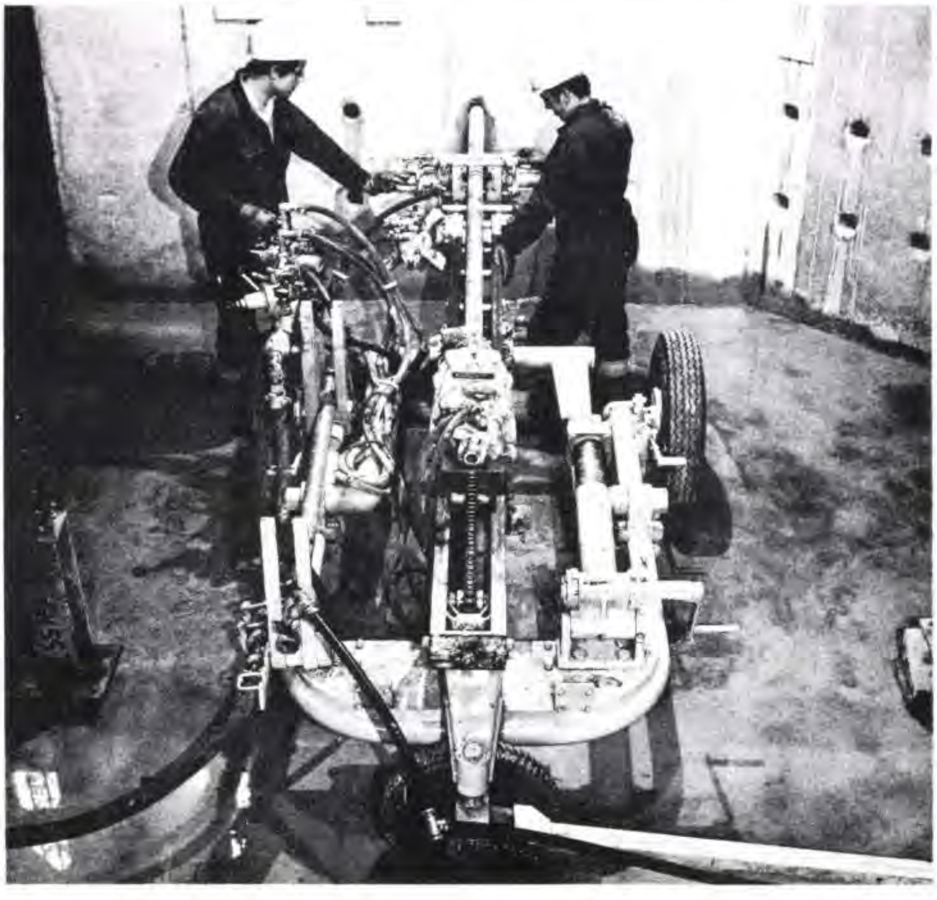


Fig. 6
North west pier.
Drilling ducts for reinforcement through foundations
(Photo: copyright Shepherd Building Group)

St. Katharine Docks

Malcolm Tucker

Introduction

The upper parts of the Port of London are in a state of rapid decline, not only because of the increasing size of ships, but also because 19th century warehouses cannot be adapted to modern methods of mechanical handling. The older docks have been closing one by one and new investment has been concentrated on Tilbury. One of the victims has been the St. Katharine Docks, which closed in 1968. For over 30 years the warehouses at St. Katharine's have been internationally recognized as important examples of functional architecture and the docks' closure has posed serious questions of their future. The Greater London Council bought the docks from the Port of London Authority for £1,800,000 and in 1969 they launched an open competition for their redevelopment, with a pretty demanding brief that included the retention of as many as possible of the listed buildings.

The winning scheme was presented by Taylor Woodrow Property Company Ltd. with Renton Howard Wood Associates as architects and Ove Arup & Partners as consulting engineers. I was involved in structural investigation of the warehouses and more recently in the detailed site investigation for Phase 1. The following account concentrates on the history of the existing structures at St. Katharine's, with a brief description of the new scheme. A fuller account of the engineering problems is given in the supplement of *Newsletter* 38. For certain historical information I am indebted to an article by my friend, Paul Carter[13].

Origins of the Docks

The early 19th century saw a revolution in the Port of London. Prior to 1800, trade was handled almost exclusively at riverside wharves, which became grossly inadequate for the increasing trade with the distant colonies. The great era of commercial dock building in London commenced with the West India Docks (1799—1802), the London Docks (1802—1805) and the East India Docks (1803—1806). These were constructed principally on undeveloped marshland beyond the eastern extremities of the City and comprised large rectangular basins ringed with austere warehouses of five or six storeys, set back behind broad quays. Besides greatly increasing the available wharfing and warehousing space, the new docks offered two distinct advantages. Firstly, with the use of locks to impound the water at a constant level, loading and unloading were unaffected by the rise and fall of the tides. Secondly, the docks were enclosed by boundary walls of impressive size, effectively preventing the pilfering which afflicted the older wharves.

The success of the dock companies was guaranteed by 21 year privilege clauses in their Acts of Parliament, giving them monopolies in the handling of certain classes of goods. Increasing trade, the high cost of road transport from the docks, and the exorbitant charges of the existing companies encouraged a group of City merchants, towards the end of the 21 year period, to consider the building of a new dock hard by the City of London. The Warehousing Act of 1823, initiating bonded, duty free warehouses, was a further stimulus to new building. In 1824 the celebrated civil engineer, Thomas Telford, was asked to prepare a scheme for the St. Katharine Docks, and Philip Hardwick was appointed as architect. Against strong opposition from vested interests, an Act was eventually obtained in June 1825. The scheme then pressed ahead with great haste, the western dock and associated warehouses being opened with celebration in October 1828, and the whole scheme being completed 12 months later.

Fig. 1
Architect's model of the proposed redevelopment. I warehouse is retained intact in the centre of the dock. B warehouse is shown in modified form, facing the Tower of London. The water area is adapted as a marina (Photo: Henk Snoek)

The roles of Telford and Hardwick

Thomas Telford (1757—1834) was already 67 years old when he was asked in 1824 to prepare a scheme for the St. Katharine Docks and the bulk of his achievements was behind him. He was able to draw on his very considerable experience, which, in the field of marine engineering alone, included the harbours of Aberdeen, Dundee and several lesser Scottish ports, and three ship canals. His commitments were, however, considerable, not least being his appointment as consulting engineer to the Exchequer Loan Commissioners under the Poor Employment Act of 1817, which involved the approval and inspection of public works throughout the country. For supervising the construction of the docks he therefore relied heavily on his resident engineer, Thomas Rhodes, a carpenter by training, who had worked for him on the Menai Bridge. In his autobiography Telford confines his remarks on the construction to praise of Rhodes and criticism of the pace with which the work had to be executed.

Philip Hardwick (1792—1870) was in his early thirties and, with his architectural practice largely confined to London, he would have given personal attention to the works. Presentation drawings for the warehouses are attributable to him and working drawings executed under his direction are at the RIBA.

Planning considerations

The chosen site, a net 9½ hectares (23 acres) just east of the Tower, was already built up. More than 1,200 dwellings had to be pulled down and *The Times* quoted the incredible figure of 11,300 inhabitants (i.e. a mean density of 500 persons per acre) as being displaced. In the 19th century such clearance schemes were regarded as of public benefit and yet no provision was made for the rehousing of the displaced population. The ancient hospital of St. Katharine, a community of lay brethren which managed to survive the Dissolution, was found a new site in Regent's Park, and its 15th century church was demolished. By chance, some drawings by A.W.N. Pugin and his assistants of some fine mediaeval work in the church are now housed in the RIBA library within 6 ft. of Hardwick's drawings of the dock warehouses. All physical remains were obliterated by the excavations for the docks.

The site is irregular in shape and constricted. Wharves along the river front were not acquired and public access to these along the southern edge of the site had to be maintained.

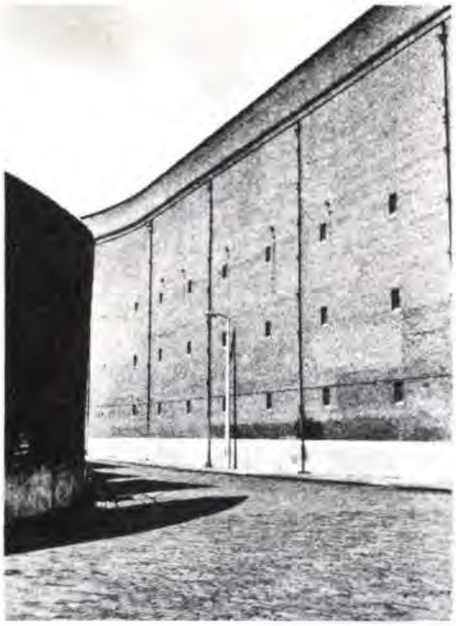


Fig. 2
Street facade of H warehouse (1852) demonstrating the impregnability of the Customs Wall
(Photo: Malcolm Tucker)

Fig. 3
The Dockmaster's House on the river front
(Photo: Malcolm Tucker)

Fig. 4
B warehouse dock facade
(Photo: Malcolm Tucker)

A government owned distillery (possibly for rum), at one time the *King's Brewhouse*, was allowed to remain, and a wooden framed building of this establishment, marked on old drawings as a 'sugar crushing house' survives to this day as G warehouse. To accommodate the maximum amount of warehousing and length of quay, Telford had to abandon preconceptions of rectangularity and symmetry which characterized previous docks. He also divided the water area, amounting in all to 4.2 hectares (10½ acres), into three parts, namely an eastern dock, a western dock and an entrance basin, thereby increasing the length of quay. The entrance basin was a not uncommon feature of docks of this period. Its purpose was to facilitate the passage of vessels in and out of the dock in the limited period of high tide, as is described later. A lock, crossed by a swing bridge, connected the basin to the river. Stop locks (single pairs of gates) separated the docks from the entrance basin, to maintain a constant water level in the docks if that in the entrance basin dropped. Telford in his autobiography suggests that one dock might be emptied of water for cleaning and repair while the other dock remained in full use, although this advantage could not apply to the entrance basin and, in any case, the stop locks pointed the wrong way. In practice, the docks were never drained, silt which was washed in from the river being removed instead by dredging. A modern engineer might have doubts about the stability of the dock walls if the water were removed.

Around the docks on three sides, six multi-storey warehouse blocks (stacks) were planned. These were built hard up against the water's edge, being supported above the quay on giant columns. This was a novel idea, partly to save space and partly to enable goods to be hoisted directly from the holds of ships into the warehouses. In the earlier docks, the warehouses had been set back behind a wide quay where cargoes were unloaded and sorted before dispatch to the warehouses. The new system eliminated double handling, with a corresponding reduction in spillages and pilfering. Cargoes could also be unloaded on to the quays, which extended beneath the warehouses, but these were somewhat congested by the supports for the structure above. Unfortunately, a ship's cargo would frequently be destined for more than one warehouse, in which case it was necessary for the ship to move its berth several times, which reduced the efficiency of the system. The principle was adopted by Jesse Hartley for his impressive docks in Liverpool (e.g. Albert Dock 1843, Stanley Dock 1857), but it was not used for the later docks in London.

On the central spit between the docks was a large two-storeyed wooden transit shed known as the King's Warehouse, probably used for goods bound for export. Most of the area beneath the quays was given over to vaults ideally suited for the storage of wines. In the north west corner of the site, facing the City, were sited the dock's administrative offices, known as the Dock House. The warehouses and the Dock House were notable works of the architect, Philip Hardwick, and are described in more detail later.

The construction of the quay walls

Acquisition and clearance of the site commenced immediately the Act of Parliament was obtained. Detailed working plans were prepared, contracts were let and excavations begun. Within a year, on 3 May 1826, the first foundations of the quay walls were laid. Excavated material, mostly clayey sand, was shipped by barge to Chelsea, where the contractor, Thomas Cubitt, was currently laying out Belgravia on reclaimed land.

The description of the quay walls in Telford's autobiography corresponds with an original drawing now at the RIBA, and has been largely verified by recent site investigations. Original levels of construction, related to Newlyn Datum were as follows: Bottom of docks: −4.3 m (−14 ft.); quay level: +5.8 m (+19 ft.); impounded level of water: +3.4 m (+11 ft.); standing water-table +0.9 m (+ 3 ft.).

Telford mentions the thick stratum of alluvial flint gravel, known as the Thames Ballast, which extends over a considerable part of the docks. This rests on the London Clay, the surface of which represents an old valley floor, and falls fairly steeply towards the river. Although in the north west corner of the docks buildings at high level are founded directly on the London Clay, in the south east corner the gravel exceeds 6.5 m (21 ft.) in thickness and extends some 1.2 m—1.5 m (4 ft.—5 ft.) beneath the base of the quay wall. It was necessary to prevent the water impounded in the docks from draining away through the porous gravel.

Below the footings of the quay walls, says Telford, wooden sheet piling 230 mm (9 in.) thick, was driven into the ground a distance of 4.3 m (14 ft.), which would have penetrated the London Clay beneath. The joints in the piling were caulked for the top 0.9 m (3 ft.) A 300 mm (1 ft.) layer of lime concrete was then laid as a footing and the dock walls built thereon. The floor of the docks was lined with puddled clay and a curtain of this clay was placed also behind the walls. The quay walls were constructed of London stock bricks and are 10 m (33 ft.) high at the face. They are 3 m (10 ft.) thick at the base, reducing by a concave batter on the outer face to 1.6 m (5 ft. 3 in.) at the top, the back face being vertical. Counterfort buttresses 1.1 m (3 ft. 9 in.) square project behind the wall at 5.5 m (18 ft.) centres. For 350 mm (1 ft. 2 in.) from the face of the wall, Telford specified the best Blue-Lias lime mortar. The interior of the wall was built of flushed brickwork, bricks being laid diagonally in alternate directions, and grouted up with liquid mortar made from Dorking lime and sand, leaving practically no voids. The outer leaves of the wall were tied together at two levels by bond courses of Millstone Grit blocks, 380 mm (1 ft. 3 in.) thick. Similar blocks, measuring typically 1.3 m x 1 m deep x 460 mm thick (4 ft. 2 in. x 3 ft. 4 in. x 1 ft. 6 in.) were used for the coping. The stone was brought from Bramley Fall near Leeds.

Fig. 5
Doric columns on the quay beneath B warehouse. Note rainwater drain pipe (Photo: Malcolm Tucker)

It is not known how inflowing water was coped with while these works were being performed. A painting in the possession of the Port of London Authority shows the whole site excavated to a fair depth with battered sides and construction of the walls proceeding in the dry. Horse driven pumps must have been kept constantly at work. Telford pays tribute to his resident engineer, Thomas Rhodes, for his mastery of these problems. The end result is a completely watertight dock the most serious loss of water being by evaporation. The floor level of the vaults beneath the warehouses is 1.7 m (5 ft. 6 in.) below the present water level, but the walls adjacent to the dock exhibit only a moderate peeling of the whitewash. The present water table in the gravel is about 1.2 m (4 ft.) below the vault floor and represents the mean level of the water in the Thames. Under a 6 m (20 ft.) tidal range, the ground water level shows a sinusoidal fluctuation of only 125 mm (5 in.), lagging a quarter of a cycle behind the tides. However, the half tide Thames Barrier might raise the water table to the level of the floor of the vaults.

The entrance lock

The reference level used for 19th century dock schemes was Trinity High Water, representing the high water level of ordinary spring tides. This is now taken as being 3.5 m (11 ft. 5 in.) above the Newlyn Datum of the Ordnance Survey but in Telford's day it seems to have been a few inches lower. Water was to be impounded in the docks at around Trinity High Water Level, 2.4 m (8 ft.) below quay level and the sill of the upper lock gates was 7.3 m (24 ft.) below this.

The lock was 13.7 m (45 ft.) wide and 55 m (180 ft.) long between gates. In cross-section, the sides of the lock curved round to meet the invert, forming a three centred inverted arch like the bottom of a boat. The lower gate sills of the lock were at 8.5 m (28 ft.) below THW, 1.2 m (4 ft.) below the upper sills, and the approach channel was dredged out to the centre of the river so that the vessels of the largest draught could use the lock during the period when river level was within 1.2 m (4 ft.) of THW, i.e. from three hours before to one hour after an ordinary spring tide. Vessels of up to 1,000 tons burthen could enter the docks. Large vessels were liable to become stranded upon the bed of the Thames at low tide, the river being moderately shallow above Blackwall, and so the period of the tide during which such vessels could enter the dock was critical. Two entrance locks in parallel were proposed in the initial scheme, but only one was built. It was essential that the single entrance lock could be operated quickly and efficiently. Traffic had to be regulated and water levels controlled. This was achieved by means of the entrance basin and two steam pumps.

Before high tide, vessels wishing to leave the docks assembled in the entrance basin and the stop locks were closed behind them. Vessels passed in and out through the entrance lock for as long as the tide permitted. At high tide, vessels could sail in and out without working the lock. There might be some drop in the level of the water in the basin when the lock was being worked, but the ships at the quays were unaffected, being isolated by the stop locks. When the critical period for traffic movements had passed, the stop locks were

Fig. 6
View along the quay beneath B warehouse, interrupted at intervals by fire walls
(Photo: Malcolm Tucker)

re-opened and vessels directed to their berths. Conventionally the lock would have been filled by water flowing under gravity through sluice gates, known as paddles, in the upper gates and sides of the chamber, and similarly emptied through the lower gates. At St. Katharine's this method was considered too slow and two Boulton and Watt 80 horsepower beam engines were installed to pump water from the river via a culvert 52 m (170 ft.) long. The water could be pumped into the entrance basin to maintain the water level or directly into the lock chamber to speed the filling of the lock. With the assistance of the steam pumps, the water level in the lock could be raised 3.7 m (12 ft.) in 5½ minutes and, as a bonus, the reservoir function of the entrance basin was rendered largely superfluous. To conserve water an additional pair of lock gates was fitted at the middle of the lock so that small vessels could lock through using only half the volume of water, an important consideration at low tide. These, and the stop gates, have long since been removed.

Adjacent to the lock is the Dockmaster's House, in the elegant yet functional style of the period, with broad eaves and a bowed end towards the river. It shows distinct signs of differential settlement of a long term nature. The explanation would lie with the depth of backfill behind the lock wall. Moreover, the lock lies on or near the site of a former inlet, also known as St. Katharine's Dock. There may be some connection here with the use of loadbearing wooden piles beneath the sills of the lock gates, illustrated in Telford's autobiography. Loadbearing piles are not recorded as having been used in other parts of the site.

In the reconstruction of 1957 the walls of the lock chamber were retained, but the gate sills had to be rebuilt. The new sills, at −2.9 m (−9 ft. 7 in.) Newlyn Datum, are several feet higher than the original sills. The level of impounded water has been raised by 450 mm (1 ft. 6in.). The conventional wooden mitre gates were replaced by single steel gates, pivoted horizontally, which are lowered by cables into the bottom of the lock. The lock is

Fig. 7 above
General view of B warehouse (1828) from Tower Bridge
(Photo: Malcolm Tucker)

Fig. 8 left
Intersecting king post trusses in the roof of B warehouse
(Photo: Malcolm Tucker)

Fig. 9 below
Groined vaults on cast iron columns under C warehouse (1828)
(Photo: J. Flowerday)

now filled and emptied by gravity, but the sluice valves are worked automatically by a sophisticated system of hydraulic controls.

It is ironic that the reconstructed lock was hailed as the most advanced in Europe, yet for two years now it has lain idle.

The Hardwick warehouses

The original warehouses were designed by Philip Hardwick, who was later the architect of the lamented Doric Propylaea at Euston Station. Those around the West Dock (later known as A, B and C) were built in 1827–1828 and those around the East Dock (later known as D, E and F) were built in 1828–1829. The six warehouses, or 'stacks', were substantially similar in design, although their ground areas varied somewhat to suit the geometry of the site. They were characterized by the two-storeyed open colonnade on the edge of the quay, above which, in all but E warehouse, rose five storeys of brick. Height from quay to parapet was about 24 m (78 ft. 8 in.), and individual stacks extended up to 26 bays 143 m (470 ft.) in length (B warehouse) and 11 bays 43 m (140 ft.) in depth (D and F warehouses). They were undercut at quay level to a depth of up to 15 m (50 ft.) on the dock side. The portions that survive can be taken as typical of the remainder.

They exhibit a noble simplicity characteristic of the functional tradition, obscured, unfortunately, by 140 years' accumulation of grime, and the bricking up of many of the windows. Their outer loadbearing walls are of yellow London Stocks, under a plain Portland Stone coping. The unfluted Doric columns along the quay are of cast iron, 5.1 m (16 ft. 9 in.) high, 1.1 m (3 ft. 9 in.) diameter at the base and 38 to 50 mm (1½ to 2 in.) thick. The columns are spaced at 5.5 m (18 ft.) centres and cast iron beams span between them, clad with Portland Stone. These colonnades make a major architectural contribution. Window openings on a regular grid in the walls above have large cast iron window frames of standard type under segmental arches, while at intervals there are vertical lines of loopholes, i.e. doorways through which goods were hoisted into the warehouses. Each bay is recessed by half a brick between pilasters with a semi-circular relieving arch beneath the parapet. Along St. Katharine's Way the walls of B warehouse rise direct from the pavement, with no windows at street level giving an impregnable appearance appropriate to a warehouse holding bonded goods. This long external frontage, which might otherwise have had a bleak and unfriendly character, is broken into units of manageable proportion by re-entrant service areas. Likewise, the long façades facing the docks are subtly modulated by the setting back of certain bays a distance of 1.8 m (6 ft.) (which enabled goods to be hoisted from the quays).

The slate clad roofs, of a complicated hipped construction with intersecting king-post trusses, are hidden from view behind the parapet. Internally, cast iron columns on a bay module of 3.7 x 5.5 m (12 ft. 1 in. x 18 ft.) support conventional wooden floors, with wooden beams 305 mm wide x 355 mm deep (1 ft. x 1 ft. 2 in.) spanning the longer direction in pairs. The floor to floor height is generally 3.65 mm (12 ft.) diminishing to 3.35 m (11 ft.), and depth of construction is 690 mm (2 ft. 3 in.). Fireproof construction using brick jack-arch floors and iron beams could have been used and this was indeed done for the ceiling of the strong room in the Dock House. Expense considerations probably ruled it out for the warehouses. However, the warehouses were divided up into sections by fire walls, with heavy sheet-iron doors. These cross walls also provide lateral stability against wind loading. The use of cast iron columns for large industrial buildings had become well established by the 1820's, the first example having been in Derby in 1792. The cast iron interior columns at St. Katharine's are of a cruciform section, with web stiffeners at intervals. Web dimensions run from 190 x 32 mm to 245 x 45 mm (7½ in. x 1¼ in. to 9⅝ in. x 1¾ in.) overall according to load, while larger sections, 315 x 48 mm (12⅜ in. x 1⅞ in.) were needed where columns rise unrestrained through the mezzanine level on the quay. Cruciform columns were originated by Telford for the construction of an aqueduct at Longdon-on-Tern on the Shrewsbury Canal in 1795–6. The cruciform section has less resistance to buckling than the hollow cylindrical sections used in later warehouses but it was easier to cast. The live load capacity of the structure is about 6 kN/m² (125 lbf/ft.²) and self-weight a mere 1.4 kN/m² (30 lbf/ft.²).

The vast interiors ot the warehouses, with their slender supports, are an impressive sight, seen however by very few members of the public. Although once quite common, structures of this type are diminishing in number extremely fast. The redevelopment proposes to retain much of the outer walls of B warehouse, but the functional requirements of its new use demand that it be largely gutted. It is anticipated that a large number of visitors will be attracted to the site and one hopes that at least a portion of one floor can be preserved to illustrate this phase in the history of building. The scheme already envisages the conversion of some of the vaults into a restaurant.

The extensive vaults beneath the warehouses were intended for the storage of wine and spirits in casks. Brick tunnel vaults of segmental section are used to buttress the external walls, while the internal bays comprise intersecting groin vaults on stocky cast iron columns. The vault floors, 3.65 m (12 ft.) below quay level, are of earth or concrete, sometimes with runways for trolleys embedded. The warehouse walls are founded on conventional splayed strip footings of brick, 1.65 m (5 ft. 5 in.) below the vault floors, and the iron columns bear by inverted segmental arches on to similar strip footings.

The Dock House

This was a handsome neo-classical building containing the administrative offices of the company, set diagonally in the north west angle of the west dock, to face the City. The central bays of the stucco façade were embellished by four Greek Doric columns beneath a bold entablature, set upon a smooth-rusticated ground floor. The building was gutted during an air raid in 1940 and later demolished. Original working drawings, dated 1827–28, including some signed by Philip Hardwick, are now at the RIBA.

Later warehouses by George Aitcheson

In the mid 19th century two further warehouses were built. The architect to the dock company at this time was George Aitcheson, a contemporary of Hardwick, the latter having retired from practice owing to ill health. Aitcheson does not seem to have been an architect of particular note, but it may be significant that both he and Hardwick worked for the London and Birmingham Railway Company.

H warehouse was built in 1852 on the southern side of the entrance basin. Four storeys of yellow brick rise sheer from St. Katharine's Way with very small window openings. On the quay side are spindly two-storeyed columns of tubular section. Wooden floor beams are strengthened by miniature iron trusses suspended beneath. Internal columns are of circular section and the roof is of wood.

I warehouse replaced the King's Warehouse on the T-shaped central spit between the docks. It was built in 1858–60 and its architecture is more ornate, including an Italianate campanile, while it is structurally much less substantial than the earlier buildings. It is notable that almost the entire ground floor is open, the walls resting on iron columns 320 mm (12½ in.) in diameter and 2.1 m (6 ft. 10½ in.) high. A cartway, 5.5 m (18 ft.) wide and two storeys high, runs within the building. Floors are of fire-proof jack arch construction on riveted iron I-beams on circular columns. It is not known whether the nicely detailed light iron roof trusses are original.

In 1853–55 mezzanine floors were inserted behind the Doric colonnade of C and A warehouses and may still be seen. Cast iron beams with fish-belly flanges rest on brackets ingeniously clamped to the cruciform columns. The soffits of the wooden floors exposed above the quays were fire-proofed at some time with hollow clay tiles. The quays themselves were paved with large iron plates.

George Aitcheson died in 1861 and was succeeded as company architect by his son of the same name. However, no more buildings of significance were erected at the dock. George Aitcheson Junior had a successful architectural practice in the City and the West End, designing *inter alia* some of the first iron-framed office buildings.[20]

Ancillary equipment

Telford's cast iron swing bridge across the entrance lock has been replaced by a plate-girder structure of 1895. However, an original footbridge, probably to Telford's design, crosses the entrance to the eastern dock. It is in the form of two cast iron cantilever trusses which retract longitudinally into the quays by means of rack mechanisms. The Boulton & Watt engines have also gone. Winches once used for the manual operation of the lock gates have fairly recently been removed. Cast iron bollards associated with the entrance and stop locks are embossed with the words St. Katharine Docks 1828. Bollards were not used for mooring, rings being recessed in the quay walls for this purpose. An interesting feature is the chamfering on the top of the quay wall in certain places to accommodate the bowsprits of sailing vessels.

Wrought iron jibs of 430 Kg (8½ cwt.) SWL projected from the walls of the topmost floors to hoist goods up the sides of the warehouses. These and various other devices in the original scheme were supplied by Joseph Bramah. The hand winches or 'jiggers' used for hoisting were partly counterbalanced by a weight descending through the building. Goods being lowered were controlled by a band brake on the winch, operated via a foot treadle by a 'jiggerman' who leant out precariously through the loophole on a curved bracket at waist level.

At various later dates large cranes with slewing and luffing jibs were mounted on the walls, especially of B warehouse. They are notable for their use of hydraulic power. Water under a pressure of about 5.5 N/mm² (800 psi) from central pumping stations became available in the mid 1850's. The mode of operation is that of a hydraulic lift. Sheaves of pulleys were fixed at either end of a hydraulic ram, one on the fixed cylinder and the other on the piston, and the hoisting rope or chain was wound round these three or four times, so that a stroke of ten feet, for instance, would move the rope through 60 ft. or 80 ft. Not only the hoisting, but also the slewing and luffing, were accomplished using such rams. The cast iron cylinders of the rams, measuring up to 300 mm (1 ft.) diameter and up to 3.65 m (12 ft.) long, are mounted vertically inside the warehouses. It would be nice to see some of these preserved in situ. Internal lifts and some of the jiggers were also converted to hydraulic power. The hydraulic machinery of the swing bridge across the lock was renewed with stainless steel rams in 1957. Another manifestation of the principle was the 'hydraulic devil', a small portable ram-winch which could be connected up to the hydraulic mains at various points on the quay, for instance to power a ship's derrick. Wooden patterns for casting spare parts for the hydraulic equipment have survived in B warehouse.

An account of the dock equipment would be incomplete without mention of the various hand barrows used around the quays. With flat platforms of various sizes, sometimes with quaint arrangements of wheels and often with attractive wooden handles, these barrows resemble illustrations of a hundred years or so ago, and are still used.

Later history of the docks

The warehouses at St. Katharine's were devoted to such commodities as wool, tea, sugar, silks, hides, indigo and ivory. Trade with Australia was strong and convicts for transportation are supposed to have been quartered in the vaults beneath I warehouse. It is, however, a reflection on the small size of vessels for which the docks were planned, that one day soon after the opening the 4.2 hectares (10½ acres) of water could boast 92 vessels. From the mid-19th century, trade began to drift away to larger docks down river, and goods were brought in lighters from there instead for warehousing. However, wine for instance was still being imported by coaster from continental ports before the last war. Latterly the warehouses were used almost exclusively for tea.

One end of A warehouse was burnt in 1937 and never repaired. The warehouses around the eastern dock were burnt out by incendiary bombs on 7 September 1940 and have been demolished except for a fragment of D warehouse and one solitary column. Fortunately the western dock fared somewhat better, for although the Dock House and part of C warehouse were blitzed, this has been successfully made good with a modern yet sympathetic office building for the Port of London Authority Police by Andrew Renton and Partners (OAP job no. 1485), which has won Civic Trust and other awards. Demolition of A warehouse for Phase 1 of the redevelopment was scheduled to start in July this year, but up to the time of writing the western dock still conveyed much of the character of the original scheme, with its disciplined architecture, essential enclosure and vital unity of scale.

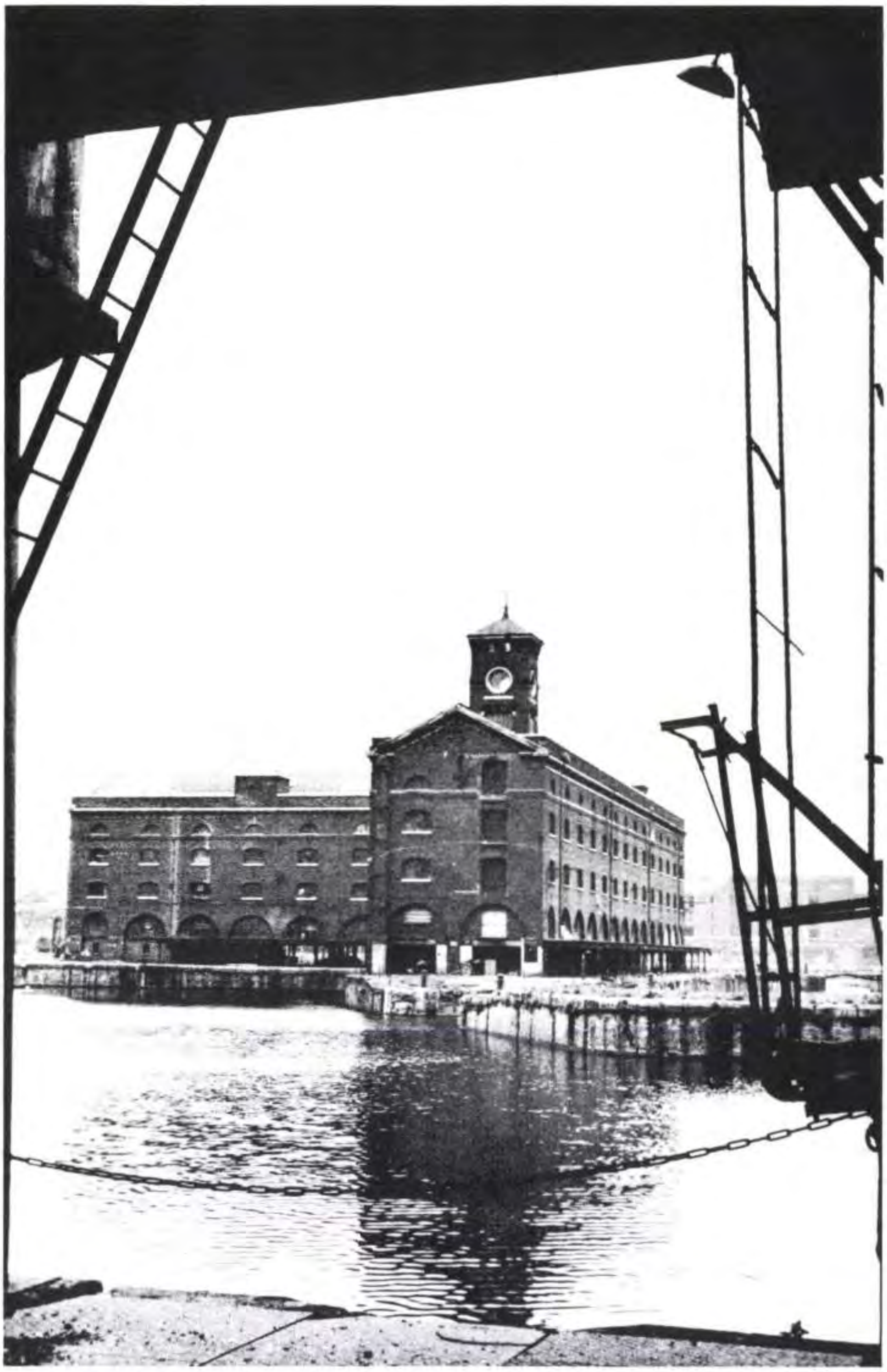


Fig. 10 left
I warehouse (1858) from the west.
It is largely open at quay level
(Photo: Malcolm Tucker)

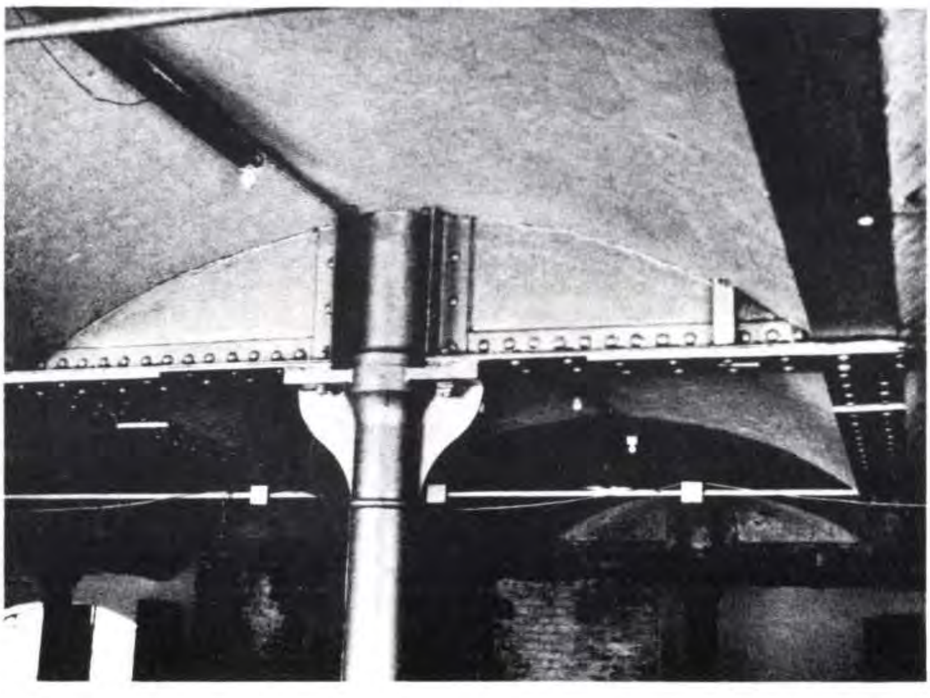


Fig. 11 left
Fireproof interior of I warehouse (1858), with brick jack arches on riveted iron beams on tubular columns
(Photo: Malcolm Tucker)

Fig. 12 below
Massive sandstone stair treads in I warehouse
(Photo: Malcolm Tucker)

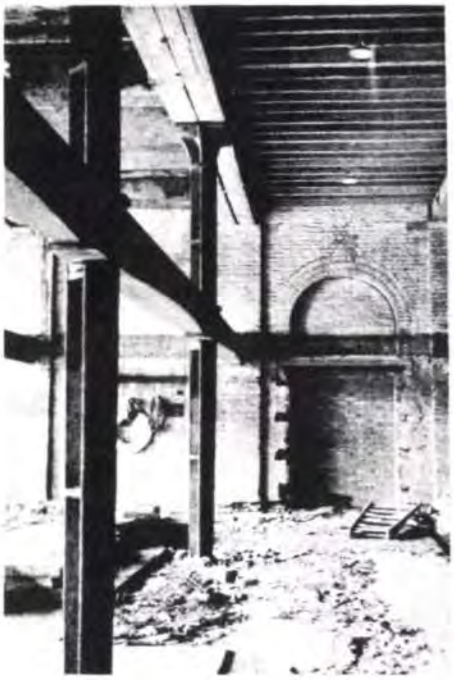


Fig. 13 above
Fishbelly flanged beams for a mezzanine floor of 1855, inserted between the original cruciform columns of C warehouse
(Photo: Malcolm Tucker)

Fig. 15 right
Cast iron retracting footbridge of 1828 at entrance to the Eastern Dock
(Photo: Malcolm Tucker)

Fig. 14 below
Cast iron clamp (c.1855) to support the later mezzanine floor
(Photo: Malcolm Tucker)

Fig. 16 below
Recess in quay for the bowsprit of a sailing ship. Note mooring rig
(Photo: Malcolm Tucker)

Fig. 17 below
A jigger rib on H warehouse
(Photo: Malcolm Tucker)

Some current uses

The area acquired by the GLC for redevelopment excludes the new Dock House and C warehouse, still used by the Port of London Authority, but includes the whole of the river frontage. Large 19th century warehouses on Irongate Wharf and St. Katharine's Wharf were recently demolished, exposing a new view of Tower Bridge. The paddle steamer, *Princess Elizabeth*, has recently been moored there for conversion to a licensed restaurant and will ultimately be incorporated in the new scheme. A bit of old dockland activity survives in a quaint two-storeyed group of buildings called Harrison's Wharf, where wines are stored. Next door, on a blitzed site, a reinforced concrete yacht is under construction.

Inside the docks, most buildings lie empty, but a thriving colony of artists and industrial designers has a short term lease of I warehouse and films are shot from time to time in the warehouses. The London Ambulance Service have a weekly training session on the site. The London Seamanship School has a small lecture room and boys venture daily out into the dock for lifeboat practice.

Redevelopment proposals

The Taylor Woodrow Group's redevelopment proposals, estimated to cost £22 m., are to be completed by 1978. The most notable feature of the scheme is the retention of the water area as a marina. When this has been deducted from the total site area, 5.75 hectares (14.2 acres) remain. A fair proportion of this will be taken up by 300 local authority houses, a primary school and community facilities, augmented by a further 320 private houses, ringing the eastern dock. Two of the existing warehouses are to be retained for a trade centre and a yacht club and this leaves the southern edge of the site for an 853 bedroom hotel of international class, a multi-denominational chapel, a complex of theatre and conference facilities called the 'Theatrevision Centre' and an under-cover sports centre. There will be a floating restaurant in the river and parking for 20 tourist coaches. 1,850 car parking places will be provided, principally underground at vault level. The traffic generated by such a high density scheme has posed its problems. No traffic is to be allowed across the middle of the dock, grade separation between road and water traffic at the entrance lock cannot be achieved, and access to the surrounding road network is in various ways restricted.

The present scheme was probably selected because it makes a bold attempt to retain some of the existing buildings. The Italianate I warehouse on the central spit is to be kept more or less as it is and converted to flats with a yacht club on the ground floor. By modification of the Theatrevision Centre the preservation of the Dockmaster's House has now also been secured. B warehouse, the only Hardwick warehouse to be retained in the scheme since C warehouse is not in GLC ownership, will undergo more extensive alterations. It is intended to convert it into an industrial exhibition centre for the benefit of City-based exporters. Its present lightweight and fire-prone wooden floors must be replaced by concrete floors and columns on new foundations. Major sections of the outer walls may also be removed to make way for service and circulation stacks, some of which may project over the dock.

Job staff are currently struggling with the design of the first phase of the scheme, the 853-bedroom hotel on the site of A warehouse, which has got to be substantially complete by 31 March 1973 to qualify for a government subsidy under the 1969 Development of Tourism Act.

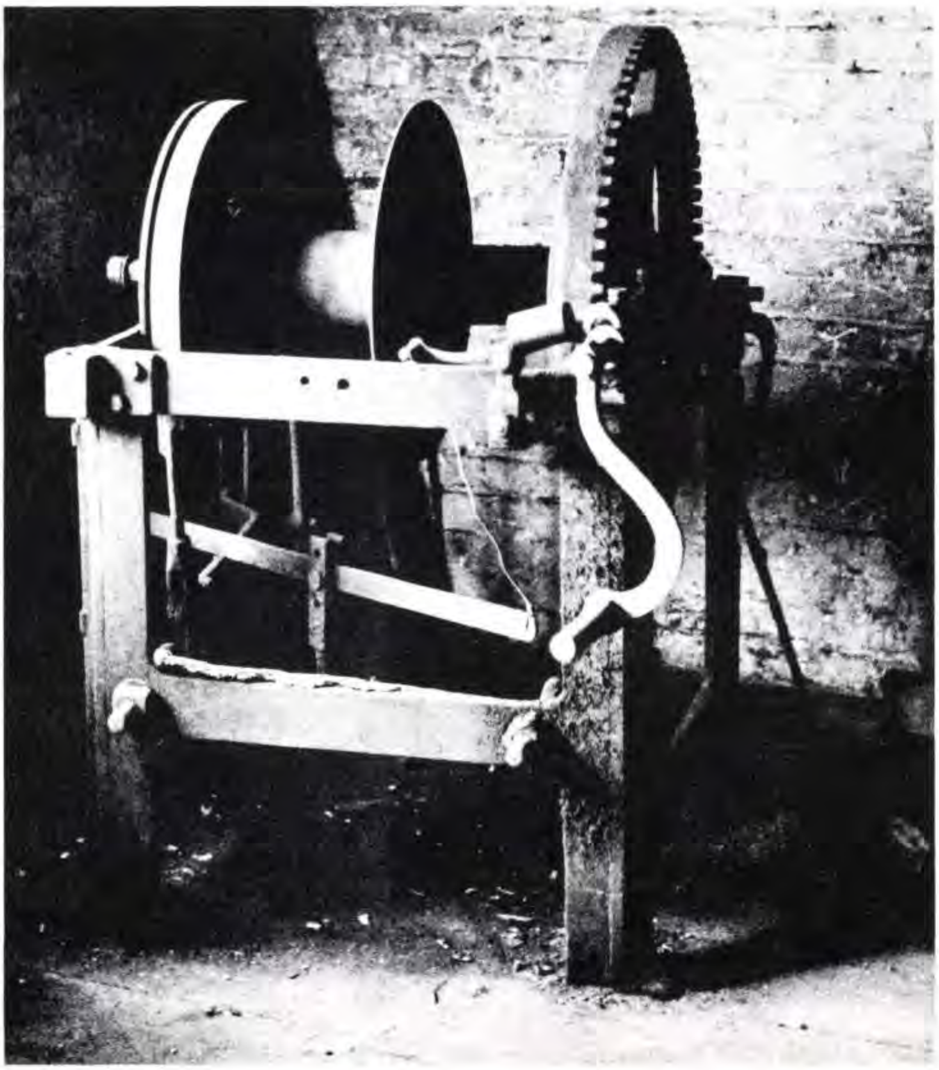


Fig. 18
A jigger winch, probably of 1828, found in A warehouse and removed to the Science Museum (Photo: Malcolm Tucker)

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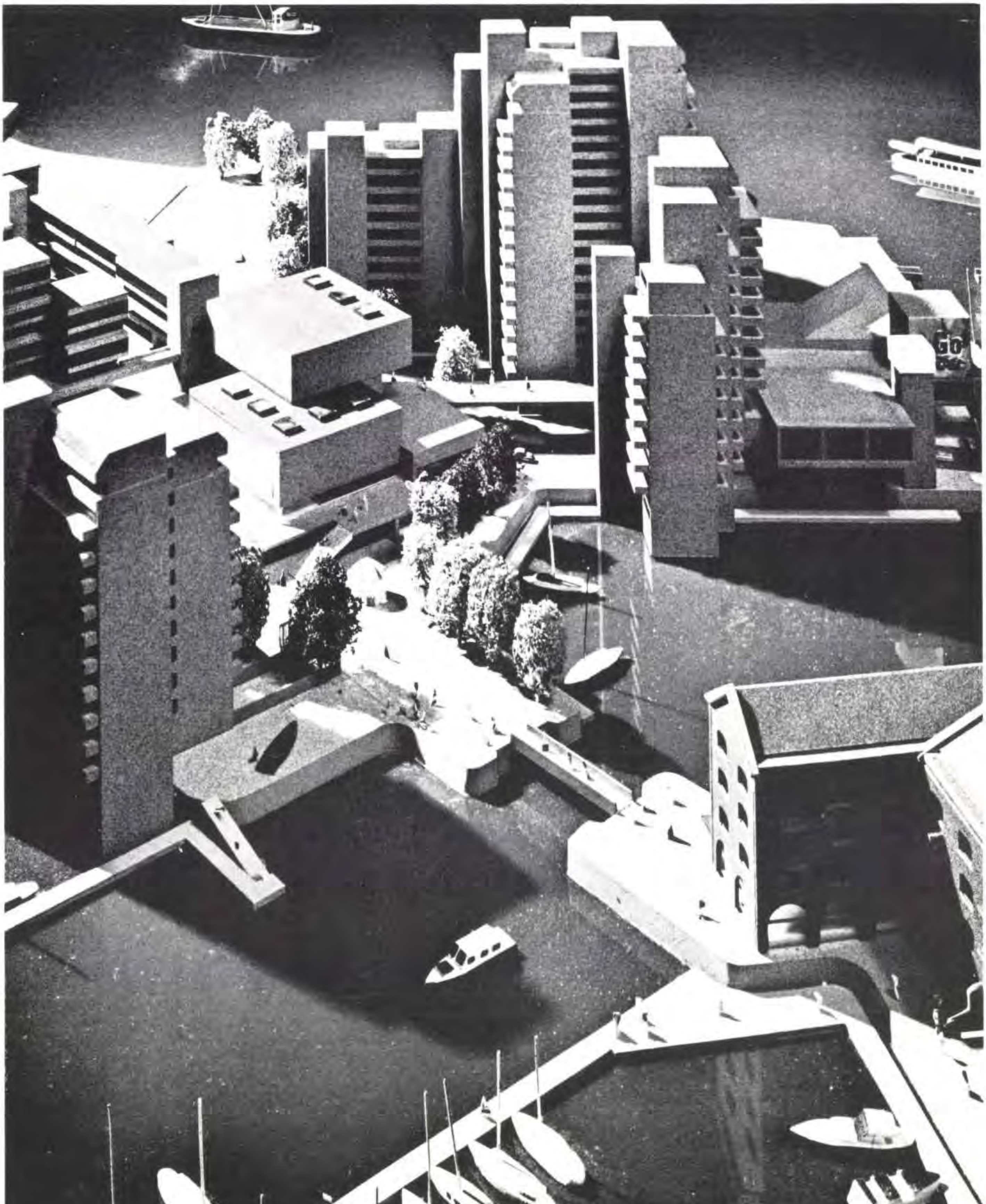

Fig. 19
Architect's model of the proposed redevelopment, showing the Theatrevision Centre (Photo: Henk Snoek)

The changing face of concrete

Turlogh O'Brien

This paper was delivered at the C & CA seminar for architects on appearance & weathering of concrete, held on 7 May 1970

The objective of this paper is to discuss the appearance and weathering of concrete surfaces in the context of the overall requirements for the building fabric and for its performance in time. The use of exposed concrete has a sufficiently long history for the likely behaviour on any new structure to be anticipated with reasonable reliability. Observation of many recent buildings suggests that, although this information is available, it is often ignored. Before making value judgements about this it is necessary to examine the particular requirements for each building. These may be circumstances in which it is perfectly justifiable to neglect weathering problems.

When considering the types of effects produced by weathering on concrete surfaces, it is clear that the distinction between changes that merely alter the appearance and those that affect 'performance' is difficult to define precisely. At one extreme, streaks of dirt may change the appearance considerably without doing any physical damage. At the other extreme, the surface may break up under weathering. In between there are a whole range of effects which may be classed as changes in appearance or as damage, depending on your point of view. Thus the stage at which 'crazing' becomes 'cracking' may vary with the type of building and with the observer. Surface flaking or loss of aggregate (from some types of finish) may be very difficult to classify.

Some people may suggest that any change in the basic surface, through the formation of any cracks, or through loss of material, should be classed as damage, whilst only those which result in streaking or staining should be classed as appearance changes. This view is not very helpful as it would imply that almost all external concrete surfaces are in need of maintenance. The alternative view would be to look at the question more from an economic point of view. Any form of maintenance of a concrete surface costs money, and the expenditure has to be justified by the results obtained. The size of the annual maintenance bill in this country has been published at various conferences, and the consensus of opinion seems to be that it is too high. Any tendency to design surfaces which are known to require high maintenance must be subjected to close scrutiny.

The techniques of comparing alternative capital costs of designs against the 'whole life' costs have been expounded in various places[1, 2], although they do not appear to be extensively used. Obviously, these techniques can be used to deal with the question of maintenance and cleaning of concrete surfaces. However, their application to this problem is complicated by the question of aesthetics. It is difficult to put a figure to the amount one is prepared to spend to maintain a particular appearance.

The relevance of all this to the subject of this seminar lies in the fact that observation shows that both the material and the way it is detailed at present do not result in maintenance-free surfaces. Yet despite this, and despite the fact that the detailing principles required to limit staining are widely known, designers often talk as if they believe concrete to be a material of infinite life and stain resistance. Speaking in broad terms the choice is as follows:

1 accept regular (five year) cleaning of concrete surfaces, and continue detailing in the way it is usually done today, or
2 aim for controlled weathering of concrete surfaces by changing the approach to detailing, or
3 develop a new form of concrete, or a new invisible long-life surface coating which resists stain formation

Observation of the extent to which clients are prepared to pay for regular cleaning of concrete surfaces suggests that only the owners of prestige office blocks are normally willing to do this. Public and university buildings may receive attention at longer intervals. Housing and schools projects are unlikely to be cleaned merely to restore a particular appearance. If this is the true situation with regard to cleaning costs, then in the short term, choice 2 (above) is the only alternative.

With the trend towards variety in concrete finishes, the architect is increasingly having to rely upon the expert advice of other people, particularly his consulting engineer and the specialist concrete companies. This trend should be welcomed, provided that the basis for the collaboration is fully understood. The specialist company must be able to deal with all queries raised by the designers. There is a trend towards competition between companies on the quality of advice they give in addition to quality and price of the product. This trend is also welcome if the companies are able to find the people with the necessary knowledge and experience to give the right type of advice. On the designers' side, the architect and engineer must know sufficient about the nature and behaviour of concrete to be able to evaluate the advice given. It must always remain the architect's job to integrate all the specialist technical advice he receives, thus producing unified design for the whole building. Contributing specialists must realize more often than they seem to that there are more problems to building design than just concrete surfaces.

Sources of information and advice

Despite the fact that concrete has been subjected to weathering for many years, the literature on the subject is not very extensive. The journals *Concrete* and *Concrete Quarterly* regularly feature examples of building with interesting concrete surfaces. Unfortunately few of these had been weathered for very long before the photographs were taken. Occasional research papers and guidance notes have appeared but they do not usually cover the topic in any comprehensive way. Building Research Station have published a book[3] which shows the changing appearance of buildings with a variety of surface finishes.

The principles illustrated for some surfaces, e.g. stone, can be relevant to concrete as well.

The architect in search of advice has a range of people to whom he can turn. The role of the specialist companies in this has been discussed above. In addition, the cement manufacturers are often able to be of considerable assistance. The role of research associations is fairly widely known. In this field the Cement and Concrete Association and the Research Committee for the Cast Stone and Cast Concrete Products Industry have established sound reputations. However, it is necessary to repeat that the advice given carries only a moral responsibility, not a legal one. A reputation for sound advice is important to most organizations, but in the end the architect has the responsibility for the design and specification.

Water flow on surfaces

In considering the effects of water flow on the changing appearance of concrete surfaces, it is essential to look at them both in terms of the surface itself and in terms of the building as a whole. The prevailing weather conditions in an area can be significantly modified by the form of the building itself. However, certain general statements may be made.

The tendency for dirt to be deposited depends on the rate at which rainwater moves over the surface. This rate is affected by:

1 height
2 orientation
3 shelter from adjacent buildings
4 shelter from parts of the same building
5 the shape of the units
6 the texture of the surface

Most people are familiar with the variation in dirt deposition that is observed over the height of taller buildings. Although the best examples are older buildings which have had time to collect considerable dirt (e.g. University of London Senate House), the effect can be seen on some quite recent buildings in the main cities. The upper parts are washed cleaner by the faster moving water.

A similar effect is observed with orientation. Once again it is the older buildings that show the most marked effect. One can find stone columns washed clean on one face, but remaining quite black on the others. The prevailing wind tends to drive the rain harder from one side on more occasions in the year, thus producing a greater washing effect. Adjacent buildings may provide shelter from driving rain, so reducing the cleaning effect. Sheltered areas are darker.

The degree of exposure of external concrete surfaces on a building may vary from severe (e.g. a roof parapet) to completely sheltered (e.g. the soffit of a concrete slab over a concourse or parking area at the entrance to a building). In between these extremes are a whole range of possible exposures, each of which will have different degrees of shelter.

Considerations of this type can lead one to question whether it is advisable to use the same concrete surface details all the way up a tall building, on different elevations, and in different relationships to other elements of the building. Although the concrete surfaces may start looking the same, they will not stay like that for long.

The use of highly shaped concrete panels as cladding is a particularly noticeable trend at present. In some cases these have been carefully detailed so that the weathering will enhance the effects of light and shade[4]. In other cases these effects do not appear to have been very carefully considered, with the result that a few buildings of this type have been cleaned after exposure for only a few years. Of course, in these cases, regular cleaning may have been intended from the start.

The emphasis on shapes for façade concrete units highlights another well-known but widely overlooked detailing problem. The main feature of concrete weathering which is normally regarded as unsightly is the differential staining that occurs through differential surface absorption and concentrated zones of fast and slow water flow. Carefully handled, this can be used to enhance the elevation. Too often it is not evaluated and the intended effect is lost. Cill details seem particularly prone to trouble on this score. In other cases the preferred water flow patterns result from the surface texture rather than its shape. A nominally flat surface will not be so flat that water will wash uniformly over it. The maxim is that flat concrete is streaky concrete. The use of textured surfaces, particularly exposed aggregate types, is partly designed to disperse the water flow and so give uniform colour change. The trouble is that this degree of texture is often not enough. A more extreme texturing is currently very popular. This is deep ribbing, either with or without broken tips. Although it has been in use for a relatively short time only, the results are looking quite good.

Of course, the traditional technique of dispersing water down a façade by using cornices, is still valid today. Current ideas for the design of buildings usually do not accept this particular trick, but it has to be recognized as a valid weapon in the architects' efforts to control weathering. By throwing the water off the façade, one does not remove it from the building. However, the wind tends to spread it out on the surface below, so that uniformity of weathering is obtained. This principle should be used much more than it is at present.

Another detail currently in use for controlling weathering, is to collect the water in channels at every level and pipe it away. Cost considerations usually dictate against complete plumbing of façades, but some buildings have been erected in which the water is piped through concrete cladding panels and discharged into open-drained joints. The water is most frequently collected below the windows so that dirt settling on the glass is not transferred to the concrete below. Obviously the deliberate channelling of water into joints puts a greater strain on the waterproofing system, and if it is less than perfect the leakage can be spectacular. Other problems arise from blockage of the plastic drainage tubes in the concrete panels. This detail is probably quite unsuitable for use on a building near trees, unless regular rodding out is done when the windows are cleaned. If blockages are not removed, severe damage due to the freezing of trapped water could result.

The overall staining of a concrete panel may also be influenced by the type of jointing between panels, regardless of whether the water is drained into the joints or not. With filled joints the water is kept on the surface, but with open drained types it is allowed in and then expelled. Current detailing of these joints encourages the water to flow forward to the face by means of 'wash-boarding' grooves and by turning the bottom of each baffle strip forward. The effect of discharging the water in this way is to encourage differential weathering between the edge of the panel and the remainder.

The problem of differential water absorption between panels and joint fillers (where these are of mortar) is well known and need only be noted here for completeness. Jointing mortars are usually more permeable, thus collecting more dirt than the surrounding concrete.

Interactions with other materials

Concrete surfaces are relatively sensitive to staining from contact with other materials. The comparison is particularly marked when they are compared to brick. The normal variation between individual bricks obscures many stains that may occur on a wall. However, this sensitivity is not so great that real difficulties are experienced in using the material adjacent to others. It is merely sufficient to require fairly careful attention to be paid to detailing.

The list of materials causing staining on concrete is quite long, even when the discolorations arising from interactions with formwork and release agents are not considered. The following notes illustrate the sources of the main problems, and serve to emphasize further the necessity of seeing any concrete surface within the context of the overall building.

1. brown rust stains from pyrites in aggregates
2. rust from scaffolding, starter bars, structural steel, badly treated bolts, tying wire left in shutters, misplaced reinforcement
3. rust stains from the soluble compounds in the early patina on cladding made from weathering steels
4. green stains from copper, flashings, bronze fixing devices, bronze sculptures
5. black bituminous stains from badly made roof trim details, from accidental spillage of waterproofing compounds on site, from bleeding of material from joint fillers of the impregnated fibre or cork types
6. stains from spillage of structural jointing materials such as epoxy or polyester resins, or from more mundane materials such as grout
7. oily staining from the bleeding of constituents of poor mastics and sealants into the pores of the concrete
8. oil and grease stains from site plant, particularly cranes
9. dirty smears around windows caused by careless window cleaning under difficult conditions
10. miscellaneous stains caused by interaction between the surface and vandals, sprayed paint being the current favourite
11. mould growth resulting from interaction with the environment in general
12. finally, deposition of salts on the surface due to numerous possible interactions, with adjacent brickwork, cracks within the concrete itself, cracks within adjacent concrete, proximity to areas treated with de-icing salts, and contact with the ground

The list is depressingly long and contains a mixture of problems arising during construction, and problems arising subsequently. With many of the very expensive concrete surfaces being used today, elaborate protection procedures have to be employed. Most of the long-term stains can be avoided at the design and specification stage. However, their avoidance requires a degree of attention to detail that it is often not possible to give. There will be many more examples of staining like those green concrete roof edge beams at Churchill College, Cambridge.

Applied finishes

The search for an invisible surface sealant that may be applied to concrete has sometimes taken on the character of a search for the philosopher's stone. Who will find the key for transmuting the base absorbent concrete surface into a noble impermeable one, resistant to the ravages of time? Must we forever use the controversial rejuvenation drug procaine (i.e. silicone) to alleviate the symptoms of ageing, subjecting the subject (concrete surface) to regular courses of the material in order to maintain the efficiency of the treatment?

Silicones act by lining the pores of the concrete, altering their surface tensions and making them water-repellent rather than water-absorbent. The surface remains clean because the dirt does not penetrate. The period of effectiveness of this treatment is variable, but there do seem to have been some advances recently. Some materials appear to work for only 3-5 years, while others are claiming lives of about 12 years. The period varies with the concrete being used, and with the environment.

Some other proprietary formulations are available which claim to act in a similar way to silicones, but have a longer life. Claims of this type are very difficult to evaluate owing to a lack of accelerated tests. Their effectiveness in the short-term may be checked, and one should not be too surprised if some of these materials fail to perform at all.

The other main way of trying to modify the surface performance without completely obliterating the appearance is to use a clear varnish. A few products are available based on methyl methacrylate. The idea with these is to form a complete skin of varnish over the surface, blocking all the pores in the process. They tend to make the surface appear permanently wet, but this, of course, can be an advantage. Once again the life expectancy of the materials is variable, and depends to some extent on how complete a seal is obtained in the first instance. The existence of pinholes through the coating can allow water to penetrate and blistering may occur later. One trouble is that with rough concrete surfaces it is difficult to obtain a complete film.

It is interesting to reflect on the fact that when concrete is painted, the necessity for maintenance is generally accepted. When clear treatments are applied they are expected to be maintenance-free. Yet it is more difficult to develop a durable clear varnish than one containing pigments.

The use of applied surface finishes does enable the weathering of concrete to be controlled in another way (i.e. other than by the careful detailing approach outlined above). It must be recognized that when they are used, a maintenance cost will be incurred (or their effectiveness will diminish) and this must be examined in the light of other claims on resources.

Quality control

Some of the construction problems involved in the use of exposed concrete have been mentioned above under staining. There are many others. The main one to be discussed here is quality control.

The requirements for quality of a concrete surface are difficult to specify. Yet, unless the attributes considered to be important by the architect are clearly described, the contractor will have some difficulty knowing just what is wanted. These attributes may include freedom from blowholes, sharp arrises, low permeability of surfaces, a particular texture and uniformity of texture, certain dimensional tolerances, the absence of crazing or cracks, and uniformity of colour both within one unit and between one and another. In some cases sub-contracts for the supply of precast concrete cladding are let on extremely minimal documents. At worst these consist of drawings showing the units required and a general statement of the type of concrete (i.e. exposed 19 mm (¾ in.) Cornish granite aggregate with white cement matrix). After approval of a sample panel for appearance, casting commences.

The difficulties of contractors in guessing what standard of finish is expected obviously affects their pricing of tenders. They may often price for 'normal' fair-faced concrete only to find later that the architect's idea of this corresponds more to the contractors' 'high quality' category of finish. It is essential for designers to recognize that there are no absolute standards in this matter. Various categories may be identified and compared with one another, and in general the same units will cost different amounts depending on the quality category.

The shortcomings of the cube test as a quality control test for fair-faced concrete are widely recognized. Unfortunately visual observation of the surface is also not enough. For good weathering on a well detailed building, a concrete surface of uniform (and usually low) permeability is required. A technique has been developed by the Research Committee of the Cast Stone and Concrete Products Industry[5] which enables quality to be assessed by the initial surface water absorption. The rate at which water penetrates the surface is measured, and compared both with other readings on the same concrete and with readings that have been obtained on a wide range of concrete surfaces.

The visual requirements are most usually controlled by the use of an agreed sample panel. For large projects this usually consists of full size panels or assemblies of panels. For smaller ones, a more limited specimen will be used. Whatever the job, the sample should include all the features that the

This series of photographs has been selected to illustrate some of the points made in the article. It is deliberately biased towards defects and failures in order to emphasise the point that it is possible to learn from experience. It is extremely rare to find the seamier side of weathering discussed openly and illustrated in this way. Most buildings only get into print when they are new and unweathered.

Unfortunately black and white photographs do not show all the subtleties of concrete weathering as changes of colour are important.

Fig. 1
Uniformity of weathering is achieved through uniform exposure and uniform absorbency of surface. The in situ board-marked concrete is contrasted with the precast exposed aggregate panels. The light streak is due to the cleansing effect of a water discharge. The colour of the wider cylinder has changed from grey to yellow-brown as the cement has been leached away exposing the sand. (About seven years)

Fig. 2
White in situ board-marked concrete fares no better in city atmospheres. (About eight years)

Fig. 3
This is another example of the superior weathering of precast exposed aggregate panels. (About three years)

Fig. 4
One technique for overcoming this problem is to paint the in situ concrete with concrete coloured paint. Of course, this looks unsightly when it begins to break down. Note the mould growth on the plinth where the detailing provides a water trap. (About four years)

Fig. 5
With older concrete the quality may be such that the erosion of the surface through acidic rainwater attack may outpace dirt deposition. The surface is self cleansing, but the appearance is still not satisfactory. (About eight years)

Fig. 6
The black and white photograph does not adequately show that the retaining wall and bridge have turned green with mould, whilst the wall of the building has turned brown due to exposure of the sand. The difference is attributable to the moisture content difference between walls of heated and unheated spaces. (About forty years)

Fig. 7
Smooth white permeable concrete weathers so badly in cities, that the appearance may be disfigured before the building is finished. (New)

Fig. 8
It is nearly impossible to patch concrete in such a way that the patch weathers uniformly with the rest of the concrete. The roof edge concrete is turning green with mould. Note also the tell-tale signs of reinforcement corrosion on one transom. (About seven years)

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architect considers relevant to the quality of the particular surfaces. It is surprising that this procedure is not used more often for in situ concrete work, where quality finishes are sometimes more difficult to achieve. With any work there is an initial learning phase while difficulties are ironed out and operatives become familiar with the procedures. How often has this learning taken place at low levels in the building (for example the main entrance area) where appearance was most important? By the time the roof is reached the quality is approaching what the architect wanted.

With in situ concrete work there is usually a clerk of works around to keep an eye on things, if not a resident engineer. If the former understands the problems of making good fair-faced concrete, so much the better. With precast work, this continuous supervision is lacking. The product is presented to the designer for acceptance. Visual checks are made and the unit incorporated in the building. Concealed defects like badly positioned reinforcement only become apparent later to the consternation of all concerned. Cover meter checks on precast concrete units should be a matter of routine, even if they are only done on a sample of say 25% of the total.

Rectification of defects

One of the most depressing things about fair-faced concrete work is the difficulty of making repairs that will both look the same initially and will weather the same. It is certainly easier to do the former than the latter, but even so, the result is often disappointing to all concerned. The main types of defects are cracks (from whatever cause), damaged or honeycombed arrises and corners, and misplaced reinforcement.

With corners and arrises, the main problem with making good is to ensure that the patch sticks and remains sticking indefinitely. The trouble is that the materials that stick well (e.g. resins) weather in a quite different manner from the surrounding concrete due to their different absorption characteristics and occasionally problems of colour stability. With some concrete finishes it is difficult to match the appearance in a patch. This is particularly true with exposed aggregate or tooled finishes.

Misplaced reinforcement presents a different type of problem. It may be misplaced from the position shown on the drawing; but still be 20–25 mm (¾ in.–1 in.) below the surface. The question arises in precast concrete work as to whether the unit should be rejected, have one section broken out and recast or be accepted without modification. The latter alternative may depend upon the contractural relationships, on the quality of the concrete in the region of the defective cover to reinforcement and on the exposure conditions once the unit is on the building. It is rare to accept recasting of parts of precast concrete units, as this presents great difficulties with the types of sections normally involved. With in situ work, the larger members often enable breaking out, re-positioning the steel by bending, and recasting to be done. Obviously it depends on the amount involved.

In cases where the position of steel only varies slightly from where it should be, the question is frequently asked, 'Is there anything with which I can coat the surface to make good the deficiency?'. As will be apparent from the section on surface finishes, this magic substance is not yet available. In certain cases, where the cover has been only 10–15 mm (⅜ in.–⅝ in.) the concrete over the bars has been removed and replaced by an impermeable resin mortar. This procedure is alright for concealed concrete or concrete that is to be painted, but it has serious shortcomings for exposed fair-faced concrete for the same reasons as given for patching corners with this type of material.

Cracks present an even more intractable problem and, until the day arrives when non-shrinking cements are available and widely used, this problem will loom large in all concrete jobs. In situ concrete usually gives more trouble than precast work, due to the sizes of continuous concrete members involved and the way they are restrained. Cracks from thermal movements or structural deflections will always be a problem.

The question 'How can I fill this crack?' normally receives the answer 'Why do you want to fill it?' as the answer to this conditions the repair method to be used. The few reasons why cracks may have to be sealed are as follows:

1 to obscure them
2 to prevent them becoming visually exaggerated by weathering
3 to prevent water penetration into a building
4 to prevent water leaching lime out, giving white deposits
5 to prevent corrosion of reinforcement
6 to restore structural continuity

Four main types of repair may be contemplated:

i cut a chase along line and fill with flexible or rigid sealant
ii rub cement + polymer emulsion into surface
iii trickle in polymer or latex emulsion sealers
iv pump in epoxy or polyester resin sealer

Before doing any of these things, it is necessary to be reasonably sure why the cracks have formed, and to decide whether they are likely to continue moving. If they will move, the only procedure is (i) using a flexible filler, and accept the damage to the appearance. If structural continuity is required, method (iv) should be used, provided that checks are made to ensure that the properties of the material really will give what is required, and that the degree of filling likely to be achieved (often quite low) will be enough. Obscuring the cracks can be done by method (ii), but the durability of the filling is questionable as very little penetration is obtained. It would easily fall out if movement occurred. If it is going to stay in it must have similar weathering characteristics to the surrounding concrete.

Polymer or latex emulsion sealers have been quite widely used for trickling into cracks. They are sold as flexible crack sealers. Figures are not available for their success rate, but it is probably very low. Much of the trouble comes from their use in cracks subject to movement, but with an extension/compression capability of less than 10%, very small absolute movements at cracks are required to fail them.

Defects that only appear after a time can be very troublesome for a different reason. On one job in London some 3000 similar cladding units were installed. After three years, four of them failed due to corrosion of reinforcement. This was traced to excessive calcium chloride coupled with very permeable concrete. The problem was how many other units would show defects in time. A number of samples were taken from two areas of the building and analysed for chloride. The two areas showed different characteristics, suggesting variability in batches of units. They also showed that the chances of having further defective units were fairly high, but that the total number of these would not be all that large. Of course, the actual units that will give trouble can only be found by sampling every unit. The client is therefore faced with the fact that he has a building on which he knows he probably has a small proportion of defective units, but that to find these before they actually fail will be very costly. All he can do is to institute a regular (annual) inspection procedure and wait for them to show up. The exceedingly unsatisfactory nature of this situation will be apparent to all designers and clients.

The question of defects in exposed concrete work is very depressing. Once a defect is there, it is nearly impossible to restore the concrete to what was originally intended. A substandard product is obtained. This really is a subject in which the cliché 'preservation is better than cure' applies entirely.

Conclusions

The discussion in this paper has ranged quite widely over design, construction and maintenance problems with concrete surfaces. Mention has been made of the vast store of information standing around us, in the form of examples of different degrees of weathering on a wide range of concrete surfaces, detailed in a variety of ways. It is tempting to think that this source can be easily tapped and the information used in future designs. In reality this will be very difficult to achieve.

The most effective feed-back procedure is for each architect to use his eyes and learn for himself. He must then develop a recall system in his mind so that his observations may be used in his design thinking. Too often, the observations are made, but not recalled when it comes to a design problem.

On a more formal basis, there is a need for a new text book, probably backed by C & CA and the Concrete Society, covering the principles of design to achieve various effects and including many examples of actual buildings. Of course, one problem that bedevils this kind of work is that the newsworthy items are those where something has gone wrong, but the architects responsible are understandably reluctant to have their design mistakes highlighted. Perhaps seminars of this type will help to change their attitudes.

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Beecham Research Laboratories: an exercise in simplicity

Alan Hart

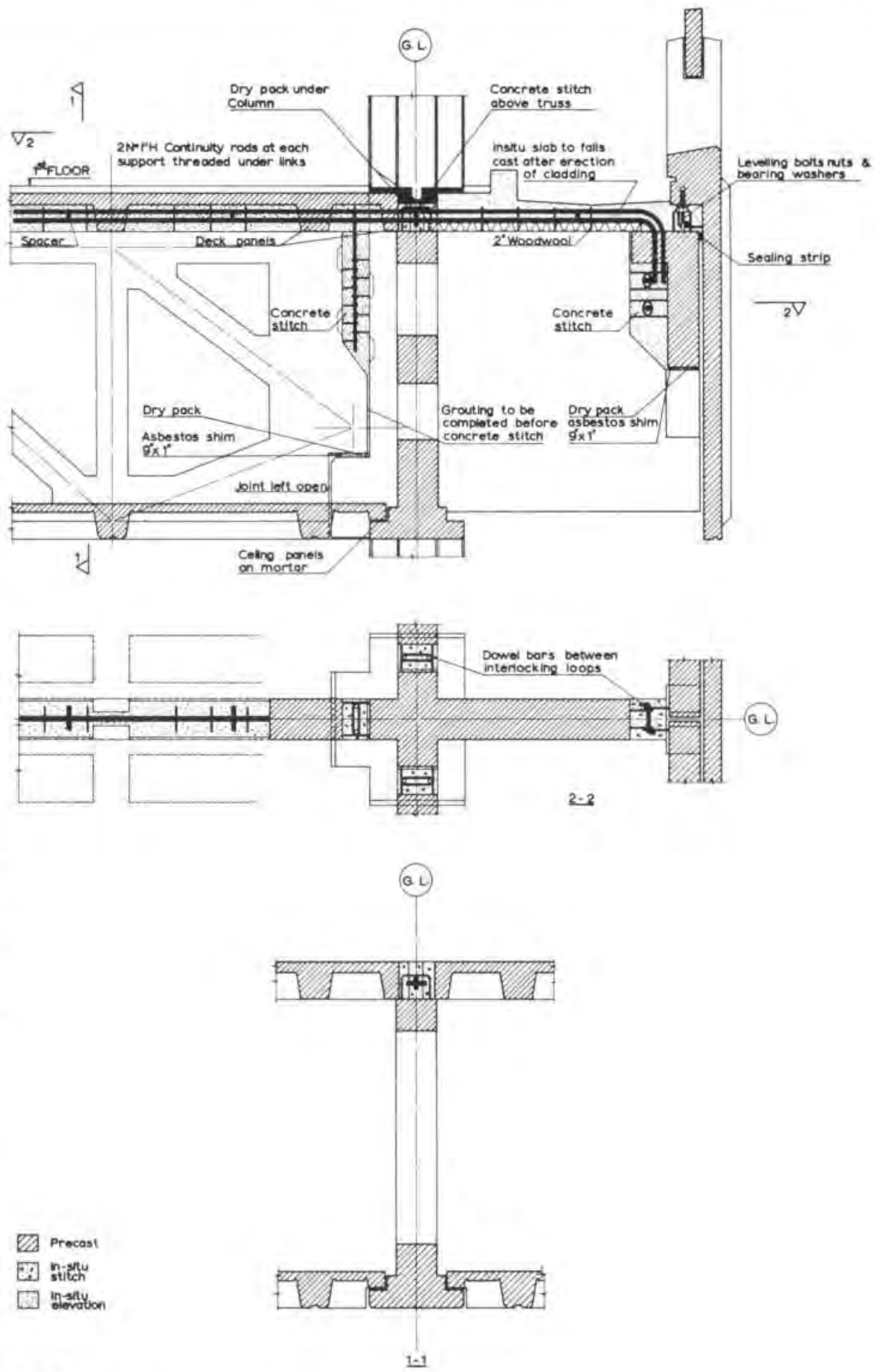

Fig. 1
Typical first floor jointing details

Introduction

Set in the Surrey countryside, facing the southern slopes of Box Hill, near Dorking, is Brockham Park, a large country mansion acquired shortly after the last war by the Beecham Group. Installed in this idyllic setting are research scientists regarded somewhat warily by local residents in the fear that a new Frankenstein is being created.

It was here, in an area where one would expect to see thatching straw rather than *Siporex* planks, that a new research laboratory was required. The actual position in the grounds was originally the stables and an old walled garden. More recently the stables have housed the scientists and their complex equipment. The contract for the new laboratories is worth approximately £1 m, of which one third represents the cost of the structure. Originally planned as a phased operation allowing rehousing of the scientists as the project proceeded, the building work went ahead in one phase and temporary accommodation was provided by the erection of *Portakabins*.

The requirement was for a building of approximately 5630 m² (66,000 ft.²) providing office accommodation and heavily serviced laboratories. To minimize interference to the occupants, services had to be easily accessible.

The structural concept

Working closely with the architect, William Holford & Partners (Canterbury office), the project gradually evolved as a structure similar in principle to that adopted by Arup Associates at Loughborough: a highly modular structure of precast components. The tartan grid chosen was based on a module of 0.9 m (3 ft.) and 230 mm (9 in.) while the main structural grid is 10.3 m × 10.3 m (33 ft. 9 in. × 33 ft. 9 in.). The building consists of two separate, two-storey blocks with an intermediate link block. Although basically a two-storey building there are basements in each block housing plant and materials. To reduce excavation to a minimum, a system of shallow basements or undercrofts was developed with deep basements where required. The relative areas of deep basement are 10% in the larger block and 70% in the smaller block. All undercrofts are deep enough to permit movement within them to check pipe runs. The substructure is entirely of in situ reinforced concrete. The structure supporting the ground floor deck is generally brick sleeper walls with reinforced concrete beams over basement areas. In the South Block, 2323 m² (25,000 ft.²) columns are carried individually on pad footings, but in the North Block, 13,252 m² (35,000 ft.²) where columns are grouped, one footing serves each group. The sub-soil on the site is mainly fissured Weald Clay.

Since complete flexibility of structure was required, an arrangement of precast concrete primary trusses on the main structural grid was adopted. The basic grid is sub-divided into areas 10.3 m × 3.4 m (33 ft. 9 in. × 11 ft. 3 in.) by secondary trusses. Except where the geometry of the block dictated otherwise, each truss spans 10.3 m (33 ft. 9 in.) and carries the loads from precast concrete deck panels on the top boom and precast concrete ceiling panels on the bottom boom. At first floor level the depth of the truss is 1.7 m (5 ft. 8 in.) and its weight is 6 tons. The depth reduces at roof level to 1.4 m (4 ft. 6 in.) and the weight decreases proportionally. Primary trusses generally span between columns and carry the loads from two secondary trusses.

The precast concrete columns are cruciform section, 530 mm × 530 mm (1 ft. 9 in. × 1 ft. 9 in.) overall, with a fair-faced finish. The length of each column was determined by the clear height required and the depth of truss supported, and each column weighs approximately 2.5 tons. In order to carry the vertical loading from the trusses, columns were cast with cruciform shaped corbels. All corbels had holes cast through to allow pipes to pass down the re-entrant angles of the column. To carry the fascia, side and corner columns were cast with cantilever fins which support an L-section precast concrete waling beam, spanning 10.3 m (33 ft. 9 in.). The fascia of each block consists of ribbed precast concrete panels 3.4 m (11 ft. 3 in.) long, weighing 3.5 tons, with the aggregate exposed, and storey height, smoked glazing. At first floor level a narrow balcony is provided and precast concrete handrails are positioned in upstands cast onto the fascia panels.

A coffered profile was chosen for deck and ceiling panels which are approximately 3.3 m × 3.3 m (11 ft. × 11 ft.) and weigh 3.25 tons. Although the basic shape of each unit remained unchanged, corner details varied and six basic shapes for each type of panel emerged. By precasting we were able to anticipate the service engineer's requirements for holes by indicating on individual unit drawings the size and location of hole required. From six basic shapes over 400 variations were obtained! The 1.7 m (5 ft. 6 in.) deep void between ceiling and deck is easily accessible and the service engineer was presented with a large area with few constraints.

There is a second, less heavily serviced void between the ceiling to the first floor and the roof. Access between floors is provided by an oil-draulic lift in the South Block and stairs in both blocks formed of in situ concrete spine beams with precast terrazzo treads. In the link block there is a spiral staircase, approximately 2.4 m (7 ft. 9 in.) diameter, formed entirely of precast terrazzo units.

Manufacture of components

All the precast components were manufactured at the Southampton works of Reema Construction Ltd., who were also the nominated sub-contractor for the erection. Units were transported to the site by road and, with the exception of deck and ceiling panels, no special arrangements had to be made. The dimensions of the latter were such that they were classified as a wide load and required a police escort for sections of the route. The consequent delays on site were reduced towards the end of construction by stock-piling units.

Few problems were encountered in casting the various units although the geometry of some was complex. To standardize the manufacture as much as possible, similar members in different trusses were made dimensionally the same. The trusses were cast on their side on a steel palate which could be rotated to a vertical plane. The weight of trusses was such that they had to be cast outdoors and lifted from the mould by a track-mounted tower crane. With two moulds in operation the production was a continuous process involving a 24-hour cycle. As the units were cast outdoors it was not possible to have a refined curing procedure, but it was possible to withdraw the steel formers after only a few hours and to lift the finished truss from the palate the day after casting. Only one truss presented any problem—a primary truss with a corbel, capable of carrying a secondary truss, on each face. In this case it was necessary to leave out the concrete round the area of the corbels and to form them when the truss had been lifted from the mould. With some complex reinforcement details in the trusses, steel fixing provided some problems during the first castings but as production proceeded it became possible to form the complete reinforcement cage and position it in the mould as a single unit. Due to a combination of hot weather, a hot steel palate and warm concrete, it was found that the top boom of the trusses was susceptible to cracking. To some extent this was controlled by adding extra reinforcement, but the problem was not considered critical. Lifting sockets were cast into the top boom above the penultimate vertical members.

With the exception of those columns with cantilevered fins, the casting of the cruciform section was quite straightforward. We had assumed that the cantilever truss would be quite separate but at the suggestion of the manufacturer it was made integral with the columns to reduce erection times. Columns were cast on their sides in a steel mould. Those columns with a fin were cast with the fin flat and where two fins were required the second was added when the concrete had gone off and it was possible to rotate the column through 90°. The arrangement of holes for lifting purposes was such as to allow the column to be lifted vertically.

Although the deck and ceiling panels were basically similar with 150 mm (6 in.) wide ribs in two directions at 1.1 m (3 ft. 9 in.) centres, they were not cast from the same mould. To obtain the desired appearance on the underside, ceiling panels were cast in a fibre glass mould, whereas deck panels were cast in timber. To standardize panels in the event of services being added or subtracted, each ceiling panel had conduit cast in vertically at the rib intersections and each deck panel had *Uni-struts* cast into the underside of the ribs.

Lifting eyes were cast into the top side of the units at the four corners and burnt off on site. Production of these units was fast since their size meant that they could be cast indoors and the reinforcement in them was simple.

Architecturally the fascia panels are one of the more important components and it was these units that required extensive post-casting treatment. They were cast face down in timber moulds with a ribbed fibreglass base. After striking they were grit blasted and the ribs knapped. Several preliminary castings were made before the architect chose a sea-dredged gravel aggregate.

The question of tolerances being so critical it was necessary for us to examine thoroughly all the moulds before the first units were cast. In this way a standard was set and the necessity to check individual components was eliminated. Occasional spot checks were made by us to ensure uniformity. Although the specification called for certain dimensional tolerances, a proviso was added that in the event of units being outside these tolerances they would be accepted if the tolerances were not cumulative or the efficiency of the structure was unimpaired.

All the structural components were in 41.4 N/mm² (6000 lb./in.²) concrete. Where it was necessary to cast units outdoors it was found more practical to use an independent ready-mixed concrete supplier than the factory's own mixing plant.

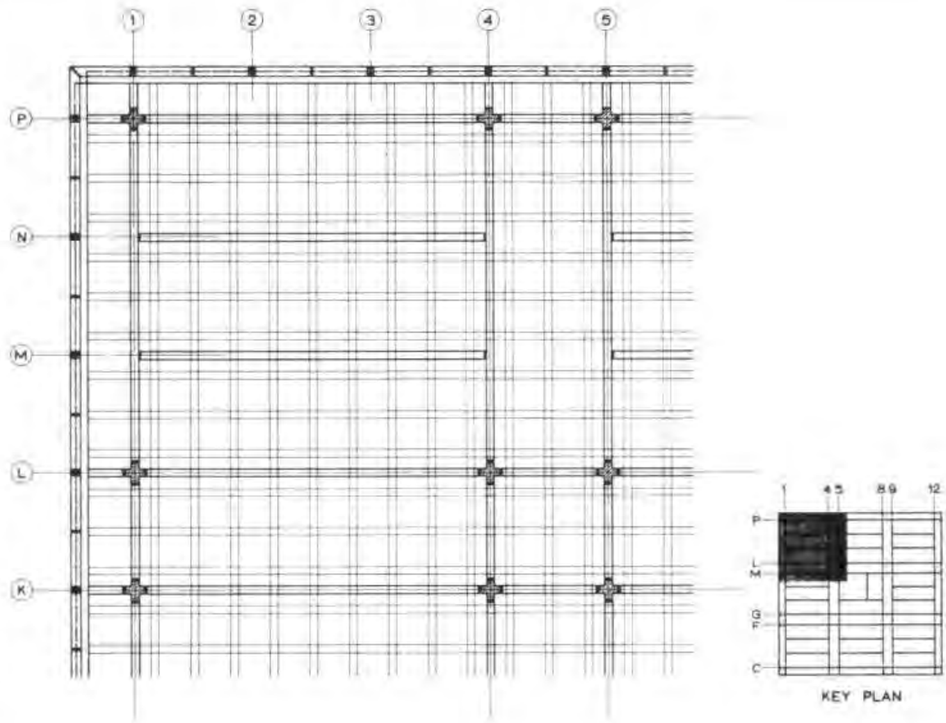

Fig. 2
First floor layout

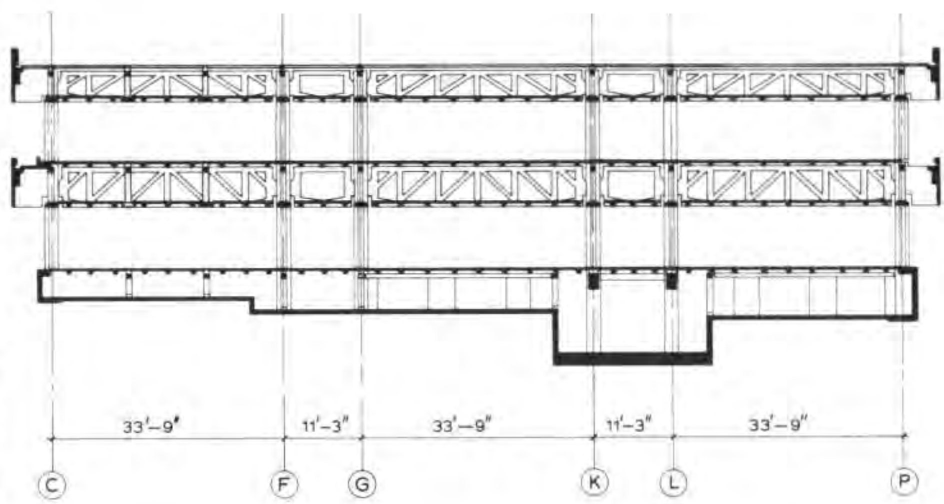

Fig. 3
Typical cross sections E-E and F-F of North Block

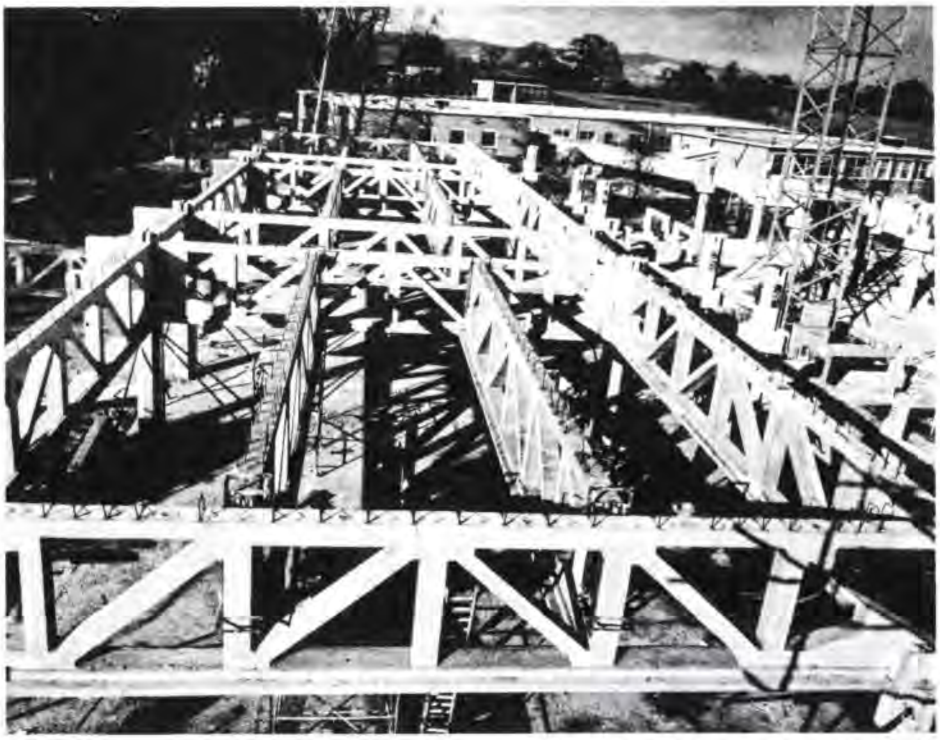

Fig. 4
North Block: layout of trusses below first floor level
(Photo: Beecham Research Laboratories)

Fig. 5
Connection detail between corner column and precast truss. Side shutters for in situ concrete stitch being fixed
(Photo: Beecham Research Laboratories)

Fig. 6
Typical connection detail prior to concreting of truss to cruciform section column
(Photo: Beecham Research Laboratories)

Erection procedure

Having established firmly the principle that the simpler the components, the quicker a project is completed, the erection of the two blocks has been incredibly swift. Had it not been for a recurrent fault in the tower crane, erection would have been completed without hitch.

Basically the completed structure can be compared to a table standing on top of another table, i.e. the continuity at the foot of the columns is negligible while the joint at the top produces complete continuity. So much of the design load is dead weight that we were not too concerned with this concept.

On completion of the substructure a thorough check was made of the compatibility of the in situ work to accept the first precast elements. Since all deck panels are finished with a 38 mm (1½ in.) thick floor screed, tolerances were reasonably liberal. Each panel was bedded on 1.6 mm ($\frac{1}{16}$ in.) thick shims of *Hyload* placed under all the rib seatings. To reduce any live feeling in the panels, an in situ concrete stitch was made along the top of the supporting beam. Erection of these panels was dependent on their availability on site, but as many as 40 were erected per day 418 m^2 (4500 ft.^{2}).

The original complex detail for fixing precast columns involved ducts cast in, projecting dowel bars and grouting up. The final detail required simply a cruciform shaped spigot cast on the base of the column and a 76 mm (3 in.) deep cruciform slot in the head of the column below. Prior to erection, tests were made on the suitability of several materials for use as shims. It was found that asbestos met the requirements and asbestos shims were used to carry all temporary loads. Asbestos packs were prelevelled centrally in each slot before the column was lifted into position. This eliminated the lengthy process of adjusting the level of the column by crane and required only that the length of the column remained constant. Each column was held in position with four push-pull props enabling adjustment for verticality.

By virtue of their size and weight, trusses proved slow to erect. Asbestos shims 25 mm (1 in.) thick were placed on the front edge of the corbel receiving the truss. The four main trusses on the grid were placed in position and clamped to the column with ply side-shutters and clamps. The next stage was to ensure that the trusses were square to one another and vertical. The narrow gap between the end of the truss and the face of the column was mortared up and the in situ concrete stitch made. Once the stitch had gone off, the two secondary trusses within the main frame were erected using the same procedure. When the stitching had been completed the frame was stable and a dry pack mortar (1:1) was packed firmly around the spigot at the foot of the column and under the full area of the cruciform. Attempts to consolidate the dry pack with a Kango hammer proved unsuccessful and packing was done by hand with a hammer and a piece of steel. A similar process was used for the 25 mm (1 in.) gap between corbel and truss, but in this case the operation was made easier by the position and nature of the gap. The dry pack takes all vertical loads and the wet stitch provides continuity and frame action. We assumed that the asbestos packs carried none of the final vertical loads.

Erection of ceiling panels was almost as swift as deck panels but more care was needed to ensure that the finish on the underside was not damaged. The nominal 10 mm (⅜ in.) gap between the panels and the bottom boom of the truss was mortared up and pointed, producing an air-tight seal.

Before placing the first floor deck panels, lengths of reinforcement were slotted through the links projecting from the top boom of the trusses and through the cruciform slot in the head of the column. Full continuity was obtained when the gap between deck panels was concreted up. In the vicinity of the column heads the level of the in situ stitch was left 76 mm (3 in.) low to enable the next lift of columns to be placed.

As erection of the fascia was not structurally important, this was erected last in the sequence. The waling beams were positioned on asbestos packs on the cantilever fins of the columns and a wet stitch and dry pack completed their fixing. Projecting bolts on the waling beams enabled each fascia panel to be located in position. To prevent the panel rotating outwards two adjustable props tied it back to the deck until an in situ stitch could be made. Although an in situ balcony was poured later the connection at each end of the waling beam was designed to resist the full overturning force imposed by the panels.

The second phase, from first floor level to roof, was erected by the same method with the exception that 610 mm (2 ft.) wide, 150 mm (6 in.) *Siporex* planks span between trusses in place of deck panels.

The main contractor, James Longley & Co. Ltd., started work on site in May 1969 and the first precast components were placed in mid-September. The South Block was completed in 16 weeks including a 5-week delay due to crane breakdown. The North Block was completed by the beginning of May 1970 in 15 weeks. These figures represent the time spent on erection of the precast units only and do not include construction of basements, semi-basements or stairs. The total length of contract is for 21 months.

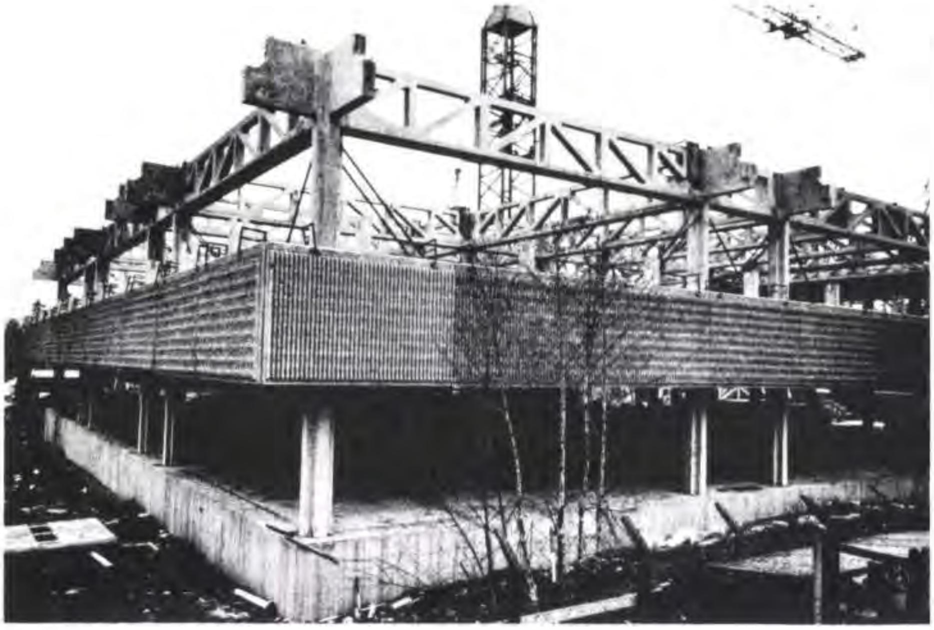

Fig. 7
North Block during erection of the frame (Photo: Beecham Research Laboratories)

Fig. 8
In situ concrete link block between North & South Blocks. Fascia panels are precast as for the main blocks
(Photo: Beecham Research Laboratories)

Conclusion

By close collaboration with the architect and the precast manufacturer, we set out to isolate as many problems as possible and solve them before erection on site began. In this way erection was controlled by production. To ensure that components were available when required entailed a detailed study of the components, the erection technique and not least, the time allowed. Had it been possible to produce only basic shapes for deck and ceiling panels the overall time for production and erection might have been significantly reduced but it would have meant additional, noisy work on site cutting holes. The short time for erection of the shell has given finishing trades and service engineers a dry and spacious surrounding in which to work.

A rigorous cost analysis has shown that certain items proved uneconomic since they serve no primary function. One must realize that the cost per item for erection, within limits, is constant regardless of size of component. Brockham Park now has, in addition to its other amenities, a building that blends perfectly with its surroundings and that is a radical departure from the stables!

Average cost of the precast frame superstructure per ft^2. – excluding fascia

Item	*North Block*	*South Block*
Supply		
1) Trusses	14/1	12/6
2) Columns	2/6	3/2
3) Deck Panels	1/11	1/11
4) Ceiling	7/4	7/4
5) *Siporex*	3/1*	3/1*
Erect		
1) Trusses	3/6	4/2
2) Columns	1/10	2/4
3) Deck Panels	2/6	2/6
4) Ceiling	5/–	5/–
5) *Siporex*	*	*
Total (excluding ceiling panels)	29/5	29/8

*Supply & Erect

Fig. 9
North Block on completion of the structural frame (Photo: Beecham Research Laboratories)