THE ARUP JOURNAL

OCTOBER 1973

THE ARUP JOURNAL

Vol. 8 No. 3 October 1973
Published by
Ove Arup Partnership
13 Fitzroy Street, London, W1P 6BQ

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

Sydney Opera House Special Issue

Contents

Introduction, by J. Zunz	2
Sydney Opera House, by Ove Arup and J. Zunz	4
Design of the Concourse, by Ove Arup and R. Jenkins	22
The glass walls, by D. Croft and J. Hooper	30
Grouting prestressing ducts, by J. Nutt	42
Adhesives for structural jointing, by T. O'Brien and J. Nutt	48
Influence of corrosion on the design, by J. Nutt	52
Sydney Opera House Awards,	54
Credits	
Editor's note	

Cover: East elevation of shell structure

Introduction

Jack Zunz

For some years we have considered writing the Opera House story. We have never done anything about it – perhaps it was lack of time, will, motivation or even the doubt that the building would ever be completed. Now, nearly 15 years after construction commenced, the Queen will officially open it on 20 October.

Instead of the book we didn't write we thought that the best thing to do would be to celebrate the end of the saga with a special issue of *The Arup Journal*. It contains some of the relatively few technical papers which have been written about the job as well as some selected photographs.

It is difficult to believe that the festivities which will mark the opening ceremonies take place 16 years since we started work on the job. After the unending technical, human and political problems, after spending over £50m, we may well ask, was it all worthwhile?

It is probably too early to say, but not too early to make some observations. Probably the most significant feature of the whole story is the astonishing reality that in a modern society, with all its checks and balances, its accountants and accountability, its budgets and budgetary controls, a folly on this scale could be contemplated. In other words, it is nothing short of miraculous that it happened at all.

In concept it is not a building of this age. It has the romanticism of former eras when autocratic patronage made great follies possible. Yet, when Utzon's scheme was chosen from more than 200 competition entries, when the Premier of New South Wales was hell-bent on starting the job without drawings, and when all those associated with Utzon caught some of the euphoria of creating one of the great buildings of the age, it looked as though the improbable would come about after all.

Much has been said and written about Utzon's concept of the Opera House. He is a man of immense imaginative gifts. In those early years he inspired all who came under his magic spell, and although there were great difficulties, they were gradually solved one by one and by 1963–64 the situation began to look quite hopeful.

But then the going got rougher and Utzon was pressed to produce drawings for the interiors. He didn't, couldn't, wouldn't, have it which way you will, and he resigned in 1966, leaving behind hard feelings, chaos, controversy, but above all a shattered dream.

Whatever judgement posterity makes about Utzon's resignation and the subsequent furore, no-one will deny his poetic, conceptual and visionary gifts and that his inability (for whatever reason) to complete the project is a tragedy. The truth is that he did walk out when information for the interiors and the glass walls was virtually non-existent. Hall, Todd and Littlemore were appointed by the New South Wales Government to the unenviable task of completing the job.

They were faced with the now fixed parameters of the distinctive roof shape, with very definite accommodation requirements which could hardly be fitted in, and above all with a half-finished work of art – and Utzon's Opera House has an artistic quality with a capital A. Some of its critics have often said that there was too much art and too little commodity. Unfortunately, Utzon is not at the finishing post to prove whether they were right or wrong and half-finished works of art can never be wholly satisfactorily finished by others.

Why did Utzon resign – did he jump or was he pushed? My guess is that he jumped. His behaviour, his letters, his interviews, all point to a path of self-destruction. He ditched his friends and collaborators for footling or no reasons at all and literally overnight left Australia never to return – at least not yet. Although Utzon's Opera House was the stuff that dreams are made of, although his use of shapes, materials, textures and colours was individual and introduced us to unique technical problems, I don't think that he ever really understood the complexity of the problems he was creating. Nor do I believe that he understood the problem-solving processes which ensued when new technology had to be developed or even when existing technology had to be adapted for new and untried forms. It is just possible that, in his seeming blindness to see that his collaboration with us was vital for the technical success of the scheme, lies another factor in his urge to leave the job.

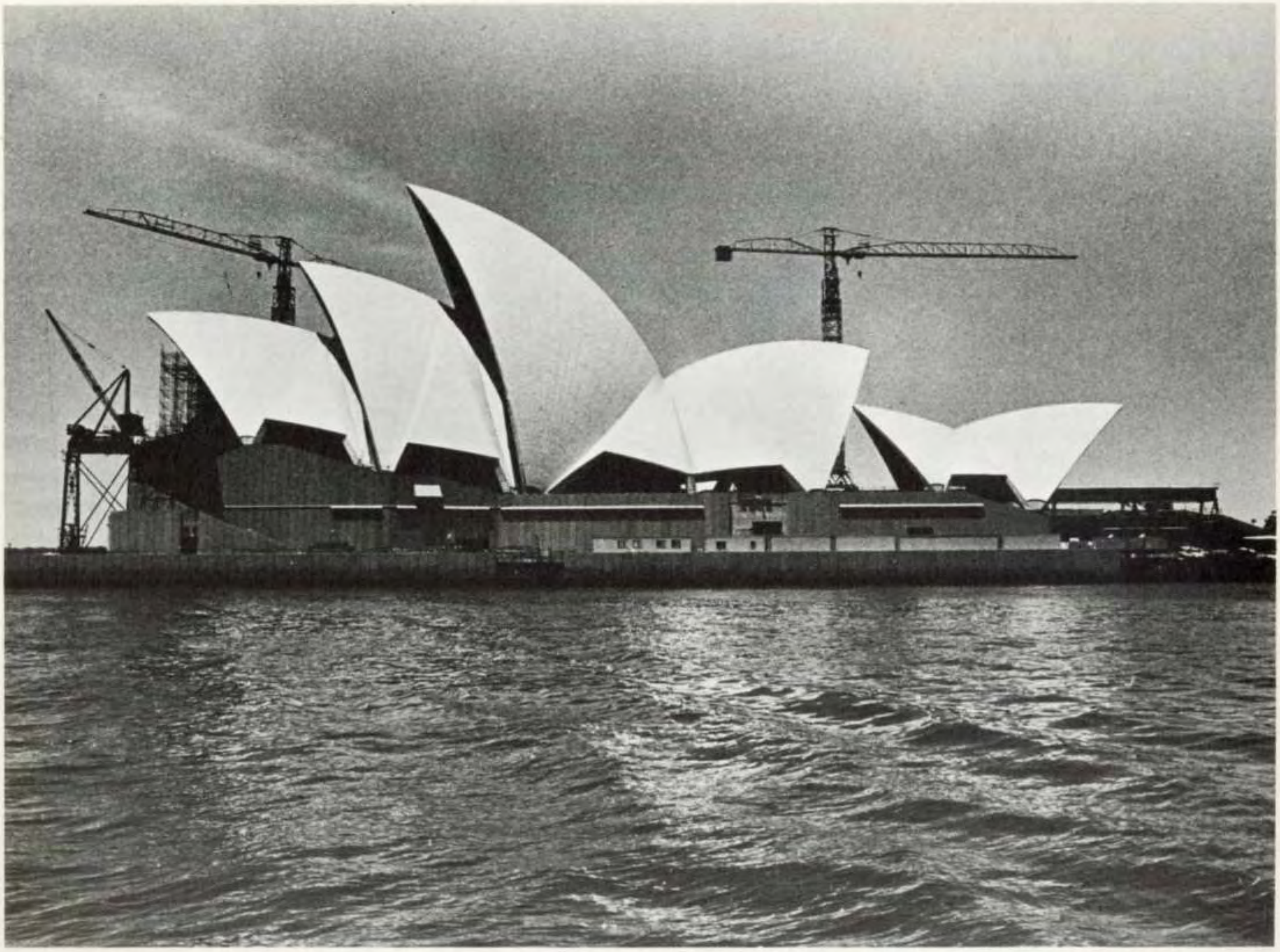
However, these are personal opinions. Despite the know-alls who have written and lectured on the subject, no-one will ever really find the truth. What is truth anyway? Whatever it is it will remain tucked away in men's minds.

Post-Utzon, the affair became more orderly, though cost estimates still kept on rocketing, but control was a little tighter and problems became more easily soluble. However, the whole thing was none-the-less just a shade duller.

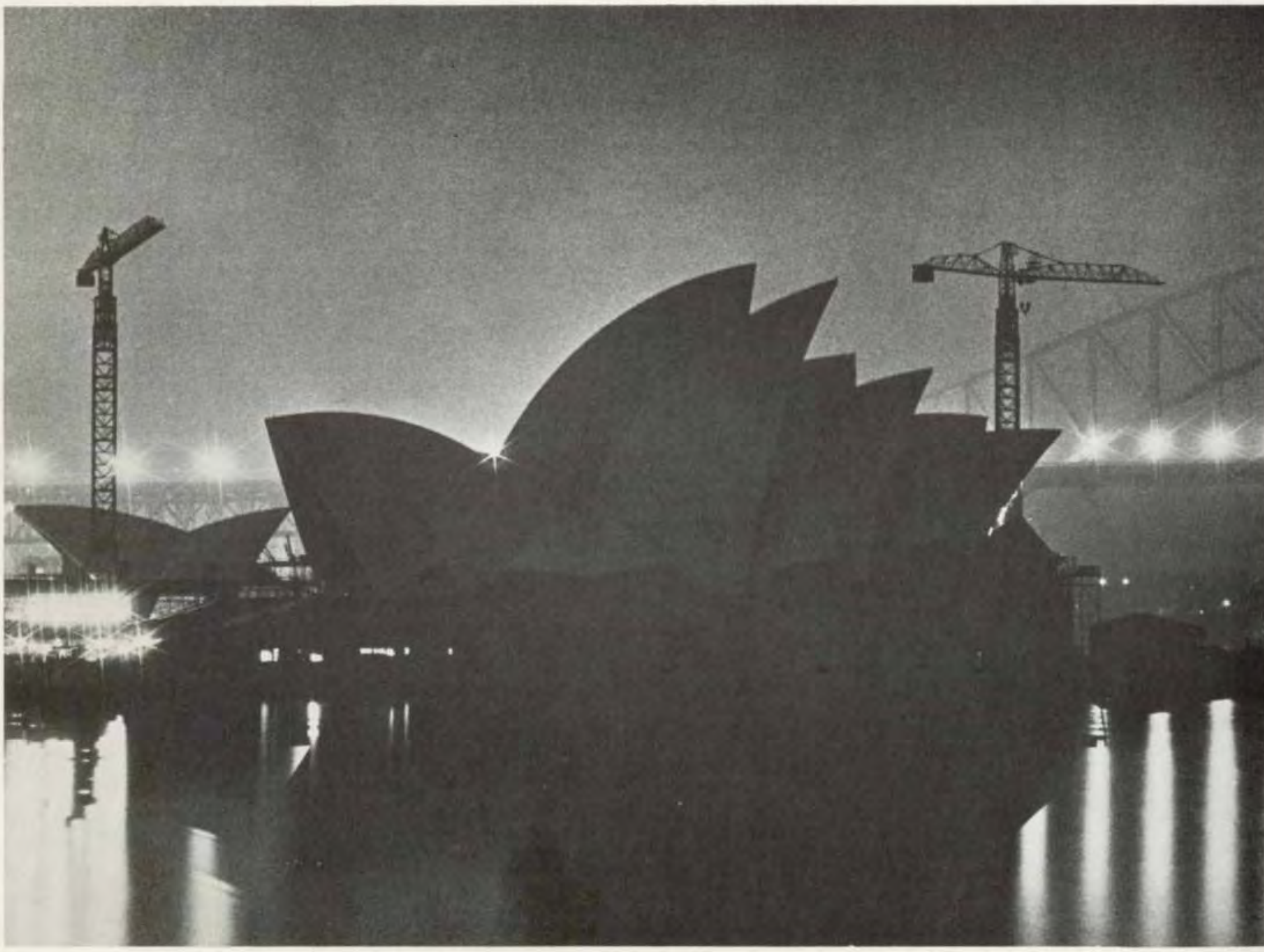
What about Arups? What has the job done to us or for us, if anything? Again, it is probably too early to see it in perspective, but there are some facts and some pointers.

Firstly the facts – we stretched ourselves to the limits of our skills. In extending ourselves and making that extra effort we developed our know-how just that little bit more. We use this knowledge in other fields. When we have been extended as much as we have, it makes our ordinary jobs easier and we hope to do them better. We have had a good deal of publicity, some critical, but mostly complimentary; we have received the Queen's Award for Industry and, if travelling broadens the mind, many of us have had opportunities for mind-stretching.

As for the more speculative consequences – we were and still are in the middle of a great controversy. Our name is inextricably linked with the building, and while its success will be linked with Utzon and his successors, its failure will reflect on us. We became unwilling pawns in the controversy. On the one hand we wanted to help Utzon and do what was best for the job, on the other we wanted to act honourably towards a client who had treated us well and fairly. Whatever we did was bound not to please everybody. So we did all and sometimes more than was asked of us and what we thought was best for the job.



(Photos: Harry Sowden)



Sydney Opera House

Ove Arup and Jack Zunz

This paper was first published in the Structural Engineer, *March 1969. It is reproduced here by kind permission of the Council of the Institution of Structural Engineers.*

Fig. 1
The site and environs (Photo: Courtesy of NSW Govt. office, London)

Introduction

The difficulty with writing anything about the Sydney Opera House is to know where to begin, what to include and where to end. There is something for everybody – it is all things to all men. It is a dream that never was, a structure that could barely be built, an architectural *tour de force*, a politician's nightmare, a population's talking point and much more.

It is not so much a building as a controversy. Controversy is news and hundreds of thousands of words have been written about it in the world's press and in popular magazines. New books are being published by well-meaning authors who generally describe, in vivid and often exaggerated and incorrect detail, the more sensational aspects of the project.[1,2,3] The cost, the length of time it is taking to complete, the use to which the building will be put and so on have been discussed almost *ad nauseam*. As a consequence the challenge, the excitement and the technical problems encountered in what must be one of the most complex structures ever built, have become obscured.

Sydney Opera House is not the kind of building which often comes within the orbit of the structural engineer. It is an adventure in building. It is not really of this age and in concept is more appropriate to the product of autocratic rule of a former era. That it has come to pass in this day and age is due to the simultaneous occurrence of many events each of low probability. Because the circumstances under which it is being built are so unusual, and because its problems are so difficult, it has created unique opportunities, both in the design office and on the site, for the development of new techniques. Many of these have since been used in more orthodox bridge and building works.

This paper will attempt to describe the structure, its constituent parts and some of the problems involved in its construction. Many problems, some highly complex, have been met, ranging from the overall concept of complex three-dimensional shapes to the details of final alignment and waterproofing. While some of these problems will be highlighted in this paper, it is impossible to include all the material of interest to the engineer and further publications have been and are being prepared.

The Sydney Opera House story is also intensely human – the stories of most major technical undertakings are – and it would be wrong to describe the structure without painting at least a sketchy background of how it all came about.

Origins

Reputedly, Eugene Goossens was the driving force during the late 1940s behind the creation of a concert hall with facilities for opera production. It was in these early years that one of the embarrassing anomalies of the whole Sydney Opera House story became established. Although it was intended to provide facilities to the highest possible standard for most musical and dramatic productions, the principal use ascribed to the scheme was and still is that of a concert hall. It would have been more appropriate to call it an 'Arts Centre', but the name 'Sydney Opera House' has now become famous and whatever those in authority may now decide, its original title will stick, at least for the foreseeable future.

The site, after some controversy about other possible locations, was chosen to be Bennelong Point. This piece of land, which supported Fort Macquarie, the sheds of Sydney's disused electric trams, juts into Sydney Harbour just to the east of the Sydney Harbour Bridge. Bennelong Point is approximately 229 m long and 137 m wide. It has a unique setting between Farm Cove and the lush green of the Botanical Gardens to the east, and Sydney Cove and Circular Quay, the maritime gateway to the bustling commercial centre of Sydney, to the west. It is also an historic site in the midst of the late 18th-century settlement of Port Jackson. The location of the site, its relationship to the natural beauty of the harbour and the Botanical Gardens, is most significant in the evaluation of the architectural merit of any buildings placed on it (Fig. 1).

The site is accessible to pedestrians from the ferry terminal, from local train services and from the commercial area nearby and it is near to all main traffic routes. The ferries could bring patrons from many suburbs located on or near the harbour.

Architectural competition

The site was chosen in the mid-1950s. The Royal Australian Institute of Architects offered its services to the New South Wales Government and it was decided to hold an international competition for the design of a complex of buildings to provide facilities for the musical and dramatic arts.

The accommodation was to include two main halls – one with 3000–3500 seats and the other with approximately 1200 seats. These two halls were to be designed for the following purposes which were explicitly stated to be in order of importance. No function was to be compromised by one of lower priority.

Large hall
(a) Symphony concerts
(b) Large-scale opera
(c) Ballet and dance
(d) Choral works
(e) Pageants and mass meetings.

Small hall
(a) Dramatic presentations
(b) Intimate opera
(c) Chamber music
(d) Concerts and recitals
(e) Lectures.

Amongst additional accommodation specified were facilities for music and drama but with smaller seating capacity.

Early in 1957 the four assessors, all architects, adjudicated the competition which caused a great deal of interest among architects throughout the world. Jørn Utzon won the first prize. He was then relatively unknown, living and working in Hellebaek, north of Copenhagen.

The assessors in their report said *inter alia* . . . 'the drawings submitted for the scheme were simple to the point of being diagrammatic. Nevertheless we have returned again and again to the study of these drawings and we are convinced that they present a concept of an opera house which is capable of becoming one of the

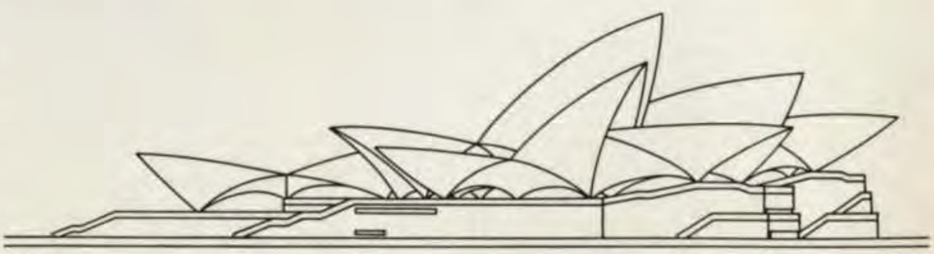


Fig. 2
Utzon's competition entry – east elevation

great buildings of the world. We consider this scheme to be the most creative and original submission. Because of its very originality it is clearly a controversial design. We are, however, absolutely convinced about its merits . . .' (Fig. 2).

There are three significant points in this part of the report. Firstly, the assessors recognized the brilliance of Utzon's basic concept despite the sparse and sketchy nature of his competition entry. The unusual roof shape was really only the outward expression of an inner plan which provided an ingenious solution to the competition problem. Whether it was possible to realize this concept in its purest form may be debated by architects for a long time to come. However, over the years of heated argument the brilliance of the original solution has often been forgotten. Secondly, for quite inexplicable reasons the name 'Opera House' had already become entrenched, despite the clear statement in the competition rules that priority should be given to concert facilities in the major hall. Lastly, the assessors clearly recognized that the design was potentially controversial. It is unlikely that they could have foreseen the extent of the controversy, the end of which is by no means yet in sight.

Utzon conceived the scheme which he submitted for the competition apparently unaided by structural engineering advice. The distinctive sculptural quality of the building with its roof structure, often likened to billowing sails, was an essential part of his first proposals. On the other hand the design was extremely sketchy and no more than an indication of the architect's intentions. The shape of the roof was based on an intuitive technical assessment of how to create surfaces with a very strong aesthetic appeal. All surfaces were free shapes without geometric definition and their structural viability had to be proved.

Strictly speaking, Utzon's intuitive technical assessment turned out to be erroneous. He had visualized the roof as thin shells. This was not possible since the very shape of the roof introduced high bending moments regardless of any structural system.

Utzon was appointed by the New South Wales Government to be the architect for the project. Ove Arup & Partners were appointed consulting engineers in the middle of 1957. An Act of Parliament, the Sydney Opera House Act, established, *inter alia*, the Opera House Trust. This Trust consists of a number of leading personalities in public and cultural life who were to act as clients under the general direction of the Minister for Public Works of the New South Wales Government. Finance was arranged through a special Opera House lottery organized by the New South Wales State Lottery.

Utzon's scheme

(Figs. 3 and 4)

Most competitors placed the two main halls back to back with the stage tower in the centre so that stage facilities for the two halls could be shared. However, this would place the entrance to the halls at opposite ends and was inappropriate for a peninsular site. Utzon placed the two halls side by side. To do so he had to forgo substantial side-wing space for the stage areas. Most production facilities had to be placed below and behind the stage, transportation being facilitated by a sophisticated stage machinery installation. Even so, he overlapped the site boundary slightly.

On his study travels, Utzon was influenced considerably by the Mayan and Aztec architecture of Central America where temples were built on platforms which formed not only an entity in themselves but also a base for the building. Sydney Opera House has such a platform, approached by a concourse of impressive scale. After ascending the steps of the concourse one will circulate around the stage areas towards the auditoria while remaining all the time in visual contact with the harbour

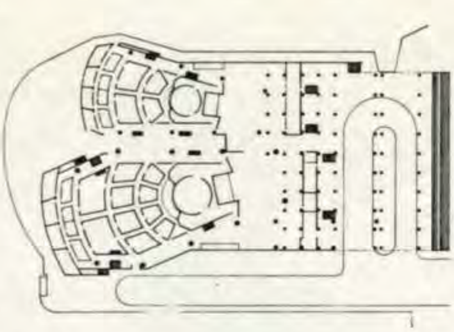


Fig. 3
Utzon's competition entry – main floor plan

through extensive glass walls in the side and end foyers. This unusual circulation arrangement obviated having the tall flytowers over the stage at the end of the peninsula where they would have been aesthetically undesirable.

There are three smaller halls tucked into the base, with numerous rehearsal rooms, dressing rooms, workshops, studios, kitchens, bars and restaurants. It is expected that if all the auditoria were used simultaneously nearly 6000 people could be seated for symphony concerts, opera, drama, ballet, chamber music, film shows, or for conventions and international congresses.

The main body of this paper deals with matters of particular interest to the structural engineer. It should not be forgotten, however, that these have become matters of interest only because of the extraordinary nature of the scheme, the unusual circumstances surrounding it, the conceptual genius of Utzon, the client's forbearance and encouragement and the apparent lack of financial constraints. These all combined to create a fertile, imaginative atmosphere for the many designers and constructors working on the project who responded by working with a zeal and a feeling that they were taking part in something quite unprecedented. That much of the euphoria has disappeared with Utzon's withdrawal from the project should not confuse the reality that the structure now standing in Sydney Harbour is the result not only of much toil and sweat but also of an unprecedented collaboration between architect, engineer and contractor.

Many architects allege that form has dominated function to the detriment of the scheme. It is not intended here to argue the case for either point of view. The only objective is to draw attention to that part of the relevant background which enables the structural engineering problems to be viewed in their correct perspective.

Geology and foundations

Bennelong Point is enclosed on three sides by sea walls constructed of heavy open-jointed sandstone blocks. The subsoil consists of fill, variable in depth and nature, which has been deposited inside the sea walls over a period of time. Water percolates freely through the fill and the joints in the sandstone blocks. Underlying the fill is Hawkesbury sandstone, which is soft to hard with variable upper layers decomposed to hard. The rock is heavily faulted and interlaced with clay seams and ironstone bands which permit the passage of water. The local authority allows bearing capacities between 131,000 and 328,000 kg/m^2 depending on the frequency and thickness of clay seams. It is obviously also necessary to judge the suitability of the rock having regard to foundation size and the depth of faults (Fig. 5).

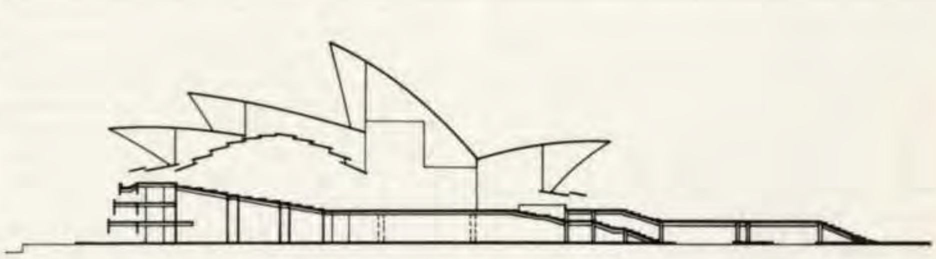


Fig. 4
Utzon's competition entry – longitudinal section

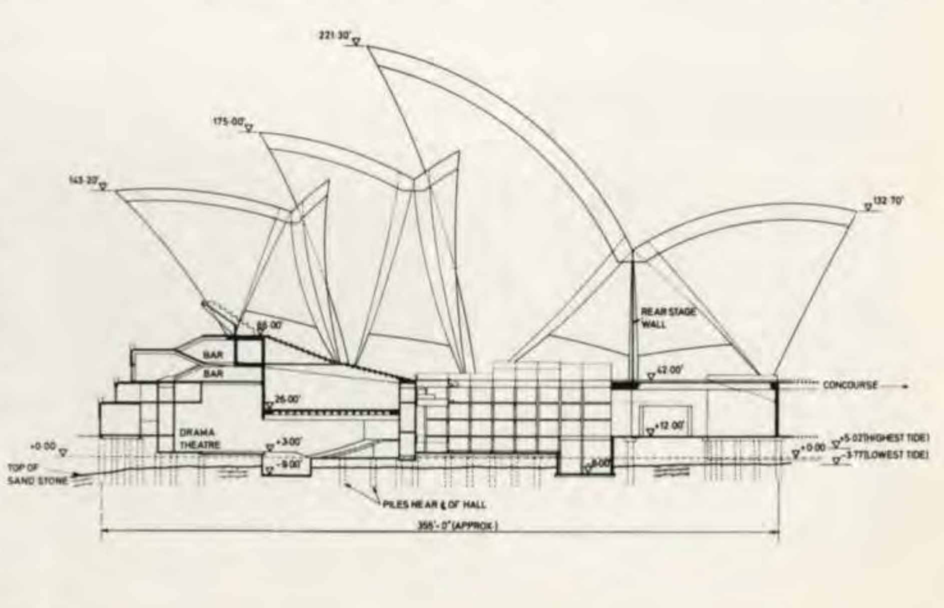


Fig. 5
Longitudinal section through the major hall

Foundations for the building generally consist of two types:

1 Bored piers, of mass concrete 0.9 m in diameter, support the building towards the north and all around the perimeter of the site where the rock falls away rather rapidly. Rock at founding level was probed and the allowable bearing pressure did not normally exceed 131,000 kg/m^2. The piers were drilled with a percussion rig. The shafts were lined with steel casings which were sealed to the rock to provide a dry hole for visual inspection and testing. Approximately 700 of these piers were installed.

2 Mass concrete foundations either in strip or pad form predominate in the central area of the building. These footings were generally used to replace fill and unsound rock. Normal reinforced concrete foundations were placed on these footings.

Rock below these foundations was probed to an allowable bearing pressure of 65,500 kg/m^2.

General practice in Sydney is to assume that settlement of foundations on the Hawkesbury sandstone is negligible for normal structural engineering purposes, provided that sufficient probing has been carried out to satisfy the appropriate statutory requirement with respect to bearing pressures. Consequently, a programme for maintaining settlement records was not considered necessary. However, the Survey Department of the University of New South Wales asked permission to use the Opera House site for an investigation they were carrying out on methods of precise levelling. Some information has, therefore, become available on the settlement of the main columns supporting the roof structure. As expected, these are of a very low order, the maximum movement recorded being 3 mm.

In view of some major structural alterations which will become necessary as a consequence of recent planning and functional alterations, investigations have been made to establish a more rational approach to assess the allowable bearing pressures on Hawkesbury sandstone. This investigation includes unconfined compression tests, tensile and punching stength tests, bearing tests (confined compression strength) and skin friction tests. It is expected that considerably higher bearing pressures could, if desirable, be utilized.

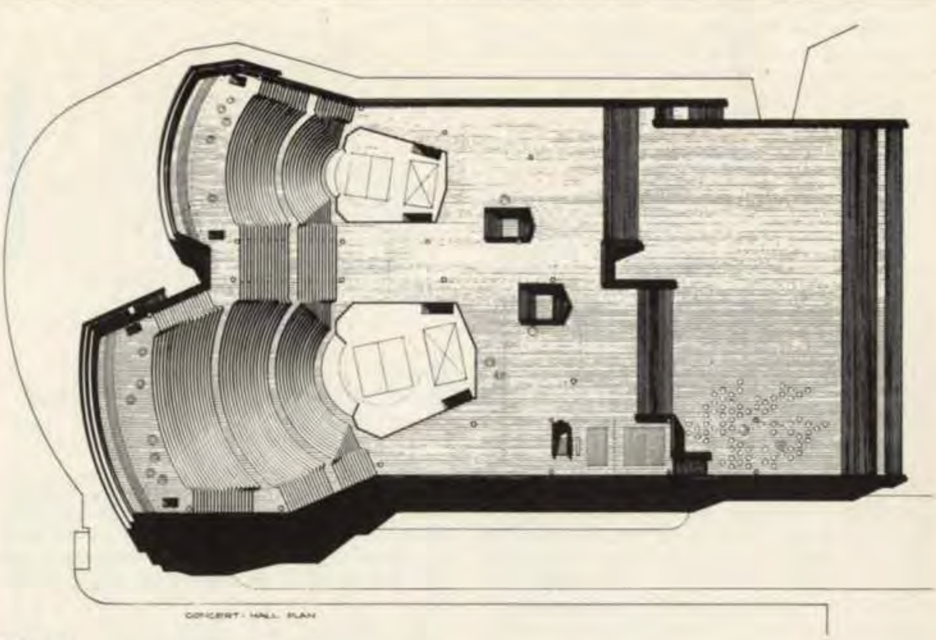


Fig. 6
The 'scooped out' auditorium seating

The base of the building

Sydney Opera House, without its distinctive roofs, is a reinforced concrete monolith surrounded on the east, north and west by a jetty-like structure called the broadwalk and approached from the south by a large prestressed concrete concourse. The monolith is almost literally 'scooped' out for the seating tiers of both main halls (Fig. 6).

The base, with the exception of the concourse, is in general constructed in normal reinforced concrete. The architect's original conception was that only basic materials, such as concrete, steel, wood, glass and, of course, ceramic tiles, would be exposed. Hence, all concrete surfaces which were to be permanently exposed were classified in three broad categories: ordinary rough formwork which was generally screened by timber walls or ceilings; boarded (101.6 mm wide) formwork with panels organized in regular patterns which was permanently exposed in corridors and other second class areas; and fair-faced formwork which was to give a true unblemished concrete surface in public areas such as the concourse and the bars where no further application of finishes was contemplated.

The structural elements are mostly walls and slabs. In some other areas, for instance backstage over the workshops, the spans are too large for flat slabs and a ribbed system was introduced. An interesting feature is the roof over the drama theatre (Fig. 7). This is hexagon-shaped, approximately 22 × 21 m. It was designed as a square grid with 762 mm and 805 mm deep ribs 305 mm wide in two directions.

The auditorium is supported on side walls and a spine beam which is carried on a torsion box at the north end. Although relatively conventional, this is a fairly complex piece of structure (Fig. 8) because the bar areas just beneath and behind the auditoria are a major feature of the scheme and are to provide clear and unobstructed harbour views. The construction of the bar area is based on a conical geometry which resulted in some interesting detail problems. For instance, ribs are rectangular in tangential cross-section only. The ribs span up to 16.8 m and are made of in situ concrete and are approximately 762 mm deep but vary due to geometric considerations. The finish is indicated in Fig. 9.

Technically, however, the most notable feature of the base is the concourse (Fig. 10). A detailed description of its evolution and design has been published elsewhere.[4] The concourse is a prestressed folded structure 95 m wide which provides the main pedestrian access to the building and covers the main vehicular access and foyer spaces. A typical section is shown in Fig. 11.

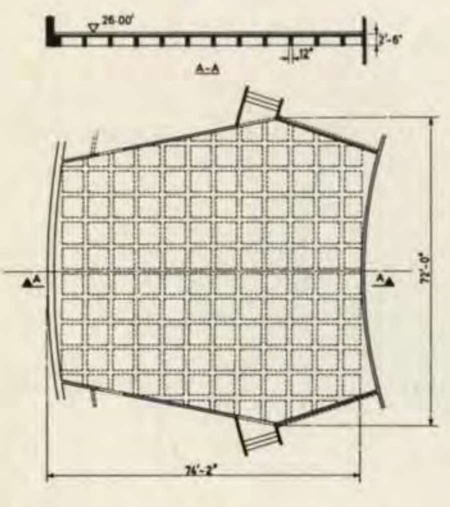


Fig. 7
Plan of drama theatre ceiling

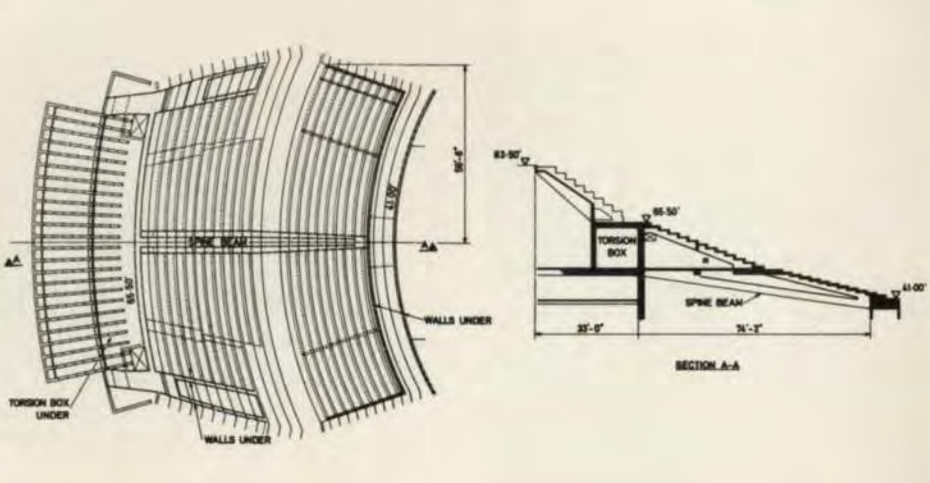


Fig. 8
Major hall auditorium floor showing main supporting structure

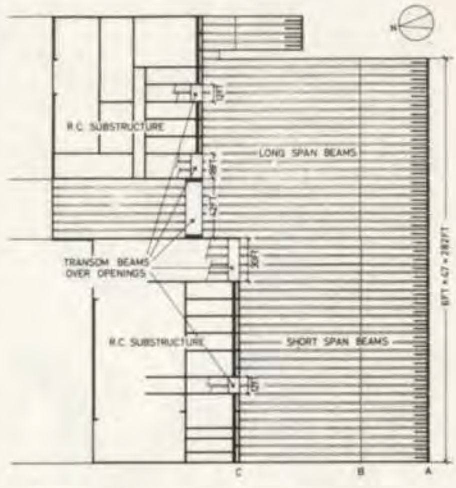


Fig. 9
Bar area (Photo: Yuzo Mikami)

Fig. 10
Concourse plan showing boundary conditions

The main spans are either 49 or 41 m. The overall depths of the units are 1.37 m and 1.14 m respectively and the plate thickness is 178 mm. Each unit is 1.85 m wide but beams were constructed in pairs 3.7 m wide, separated from the next pair by an open joint. The geometry of the beams, like that of the roof, was the result of considerable development and rationalization. That finally used, contains surfaces which intersect in sine curves and which are derived by cross-sections changing as shown in Fig. 12 such that C–F is a straight line. Typical cross-sections are shown in Fig. 13 and the geometry of one cross-section in Fig. 14. Precast slabs are placed to bridge across the units which also act as drainage channels.

The beams were post-tensioned with Gifford-Udall equipment using 28.6 mm diameter strand. Each strand had a minimum tensile strength of 807 kN. The thrust in the inclined leg of the portal is held by prestressed concrete ties. These ties connect to the reinforced concrete substructure which is in turn connected by shear walls to the concourse slabs. Because of the open joint a pair of beams could be stressed together independently of adjacent beams. The ties for each pair of beams are connected together so that the thrust could be adjusted after time-dependent distortions had taken place. Adjustment was carried out by ship jacks each of 2000 kN capacity (Fig. 15).

The analysis of the structure was conventional but computer runs for each beam included 19 load conditions which took into account jacking forces, temperature differences, as well as creep and shrinkage effects. Measurements have been taken to correlate actual behaviour with that predicted by calculations. Figs. 16 and 30 indicate views of the finished structure.

Fig. 11
Longitudinal section of concourse beam

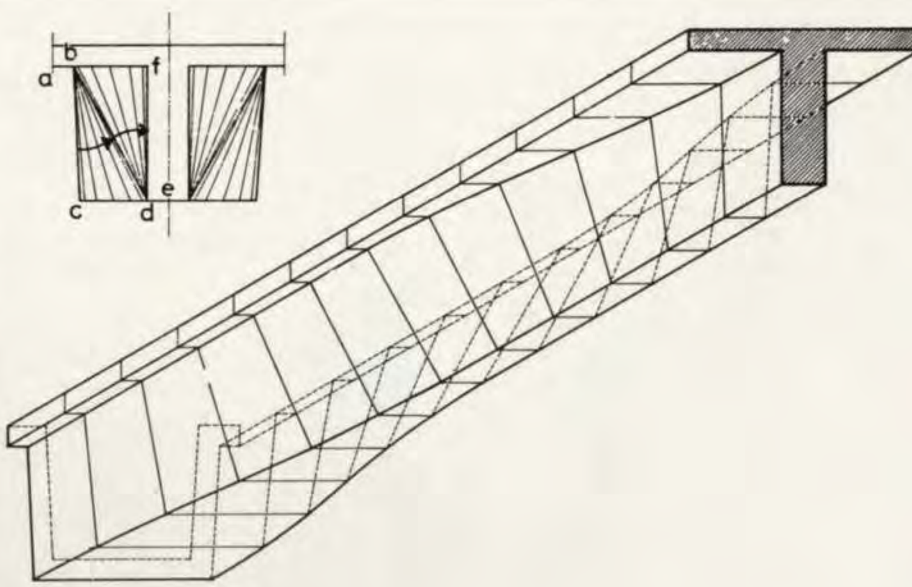


Fig. 12
Isometric view of a concourse beam

The roof structure – early development

The architect's competition scheme for the roof was that of four main pairs of surfaces for the main hall. The proposals for the minor hall varied slightly (Fig. 2). Each surface was a triangle in elevation, with boundaries formed by curves in space geometrically undefined. In cross-section, a pair of surfaces (or shells) formed a gothic or ogival arch. The main shells were connected to each other by a further series of surfaces termed 'side shells', also geometrically undefined.

The terminology used to describe the roof structure has grown with its development but is, strictly speaking, misleading. The term 'shell' stemmed from early pious hopes that membrane action would largely suffice to support the roof structure.

The structural implications of Utzon's design were first discussed with him at the first interview after he had won the competition. There were numerous factors which made it difficult to comprehend the whole problem:

1 The interplay of surfaces made an assessment of structural feasibility by normal approximations difficult and of dubious value.

2 The scale of the structure was misleading. The size of the site and the scale of the harbour and the bridge tended to diminish the building's apparent dimensions.

3 Not only were the roof shapes geometrically undefined, but external and internal finishes had yet to be chosen, the auditoria ceilings and their acoustic requirements had not been formulated nor were the size and details of the stages and machinery available.

In view of these obvious problems and others which were inherent in the architect's design, other solutions were considered. These were simpler structural forms which could eliminate some of the large bending moments inherent in the roof shapes. These proposals included

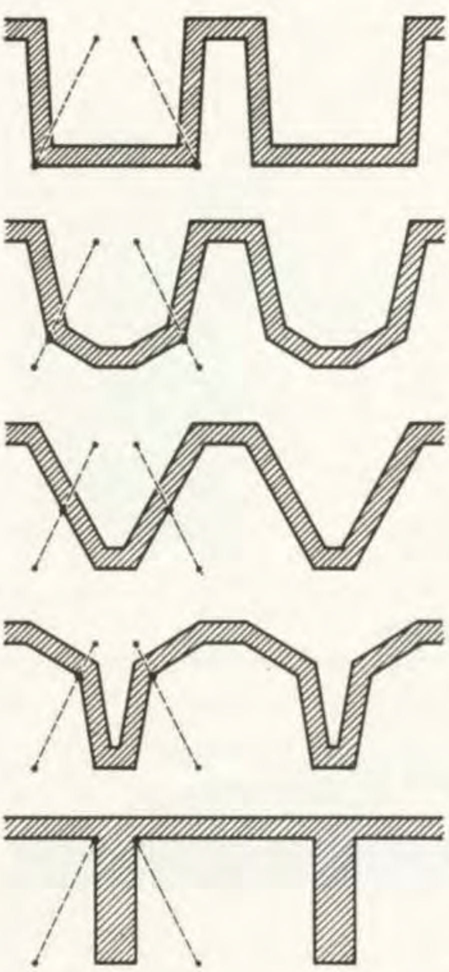


Fig. 13
Cross-section through concourse beams

non-pointed arches, doubly curved shells covering each hall or, for that matter, a single roof without discontinuities over both halls. It became evident, however, that any major deviation from the architect's proposal would destroy the essential sculptural quality of the scheme and it was decided, therefore, that all endeavours should be directed to finding a structural solution that would retain the profile and silhouettes initially conceived.

Between 1957 and 1961 analytical work and model tests were directed towards finding a comprehensive statical solution. Several geometric arrangements and structural systems were developed (Figs. 17 and 18).

In structural engineering terms the problems to be solved were as follows:

1 A geometric discipline had to be imposed on the surfaces in a way which would provide adequate clearances for the stage towers, balconies and auditoria roofs, all of which were unknown in any detail, and provide surfaces and silhouettes in accordance with the architect's fundamental concept. The discipline was also needed to predict the spatial position of points on the surfaces for construction purposes.
2 The structure had to be proved stable under all possible loads and without undue distortion under normal service conditions.
3 The wind loads on the curved surfaces were unknown and had to be established by wind tunnel tests.
4 A construction method had to be evolved having regard to structure, cladding (tiles) and the variable geometry of any staging system.

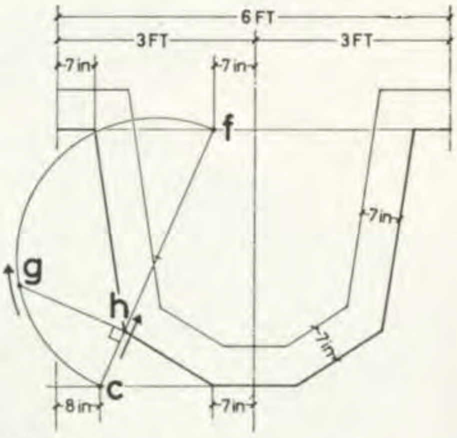


Fig. 14
Geometry of concourse cross-section

Fig. 16
View of the concourse soffit

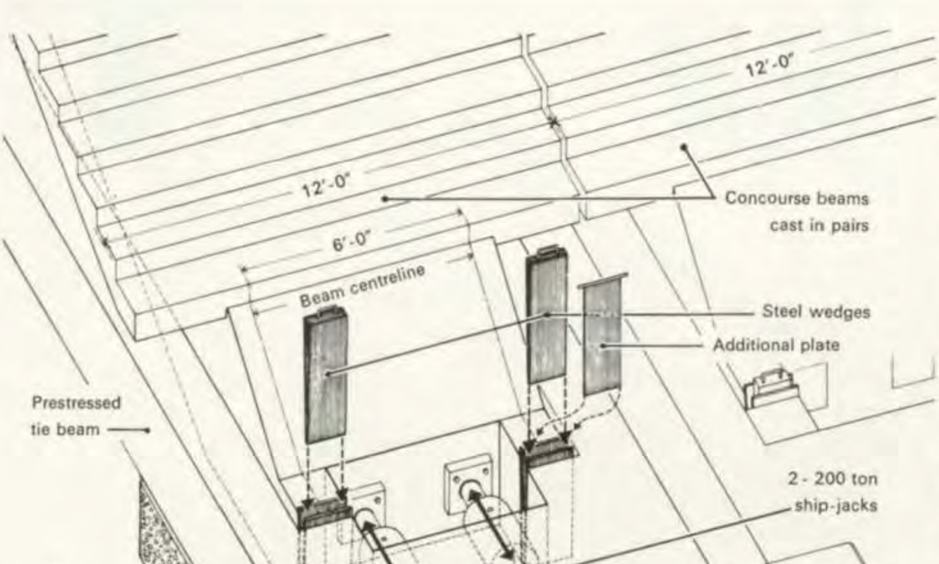


Fig. 15
Isometric view of the concourse jacking

It was clear in these early days that to achieve a solution at all, to make it possible to build the structure, extensive use of electronic digital computers was necessary. It would otherwise have been almost impossible to cope with the sheer quantity of geometric problems, let alone the complexity of the analytical work.

Early structural solutions made use of every possible source of restraint (Figs. 17(B) and (C)). All four sets of main shells and side shells were interconnected. The so-called louvre walls, the crosswalls in the opening between the two shells of a pair, were a vital part of the continuity between the shells. This structural interdependence, while necessary for stability, introduced substantial analytical complications. Theoretical researches were carried out in parallel with the model analysis to establish forces, moments, and column loads.

The architect welcomed the introduction of a geometric discipline. Early geometric solutions embodied a system of parabolas and this greatly improved the appearance compared with the original free shapes. What is most significant, however, is that the introduction of this discipline paved the way for the gradual rationalization of the design and construction and for the introduction of geometrically similar elements for factory production.

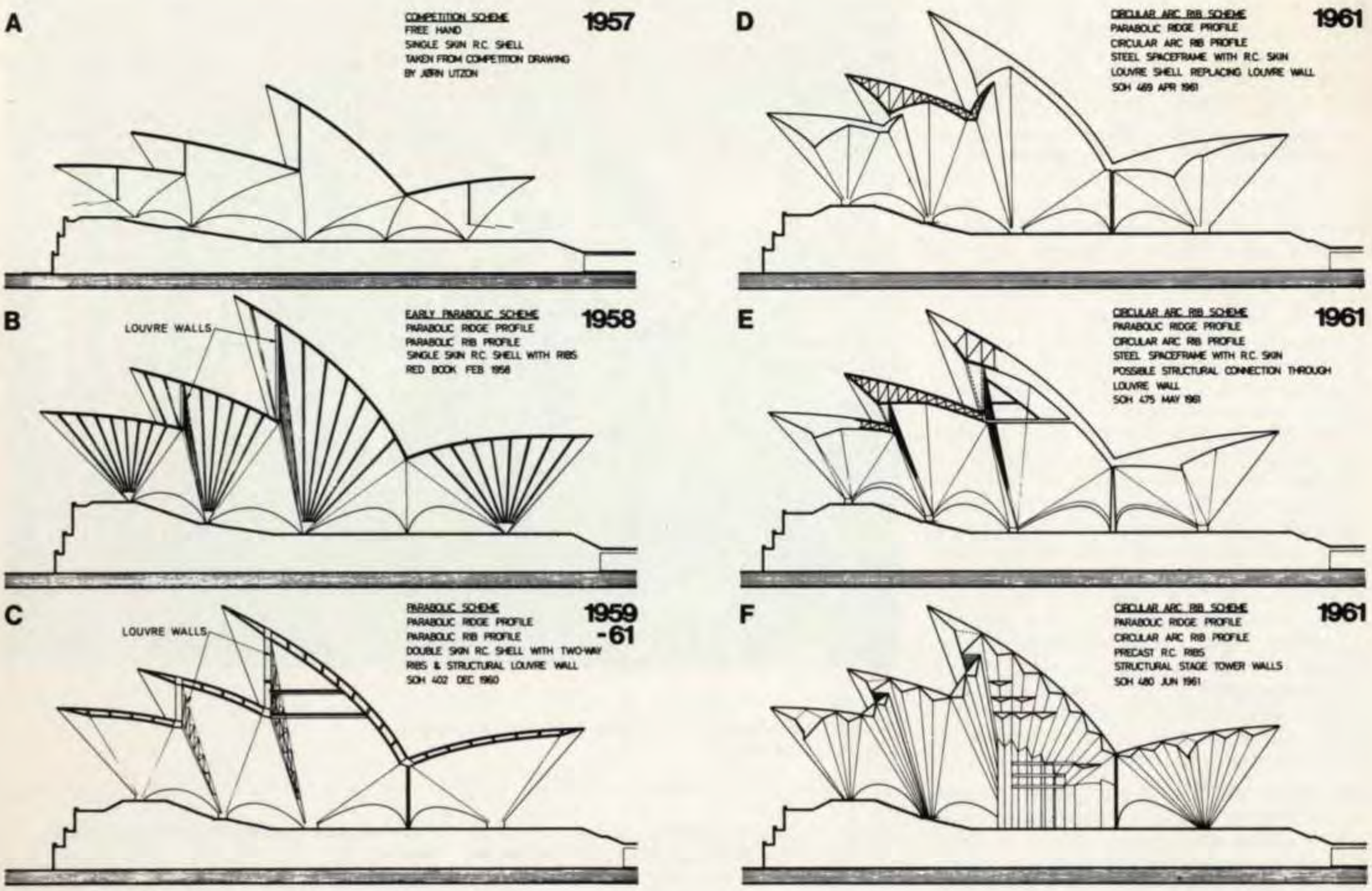


Fig. 17
History of development of roof design: 1957-1961

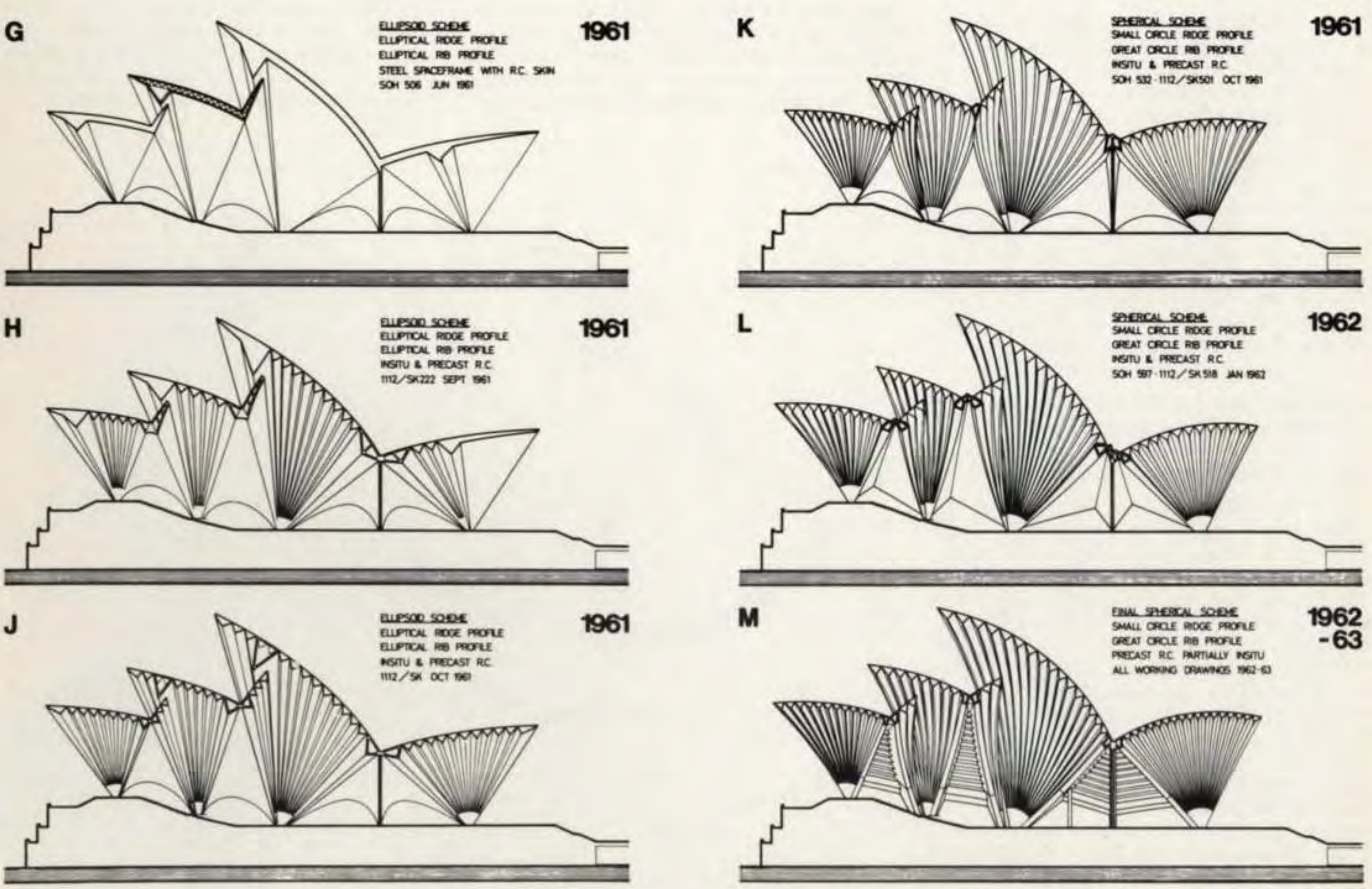


Fig. 18
History of development of roof design: 1961-1963

During this early development, the geometry underwent several changes. Side-shell geometry was changed to elliptic paraboloids and then the surfaces were changed to ellipsoids. These changes were made to improve the structural efficiency. The visual consequences were not as marked as those which followed.

The model for the laboratory tests was made to a scale of 1 to 60 (Fig. 19). The original version of the model was made in Denmark of white perspex which had less creep, slightly better elastic properties and was less susceptible to moisture and temperature changes than the transparent perspex usually used for structural models. The surfaces were manufactured by pressing warmed sheets of perspex into wooden moulds. The model was tested at Southampton University while at the same time the results obtained were collated with analytical solutions which were being developed. Dr. L. G. Booth from the University staff gave valuable advice and assistance to the engineer's team which carried out the model tests. The tests and calculations were all carried out on the rigidly interconnected scheme based on the parabolic geometry. As the tests progressed, the scheme underwent developmental changes. Early studies were based on a single (reinforced concrete) skin. As the magnitude of the bending moments became apparent, the scheme was changed to two shells approximately 1.2 m apart. The web between was capable of transmitting shear forces. In parallel with these investigations, wind tunnel tests were carried out.

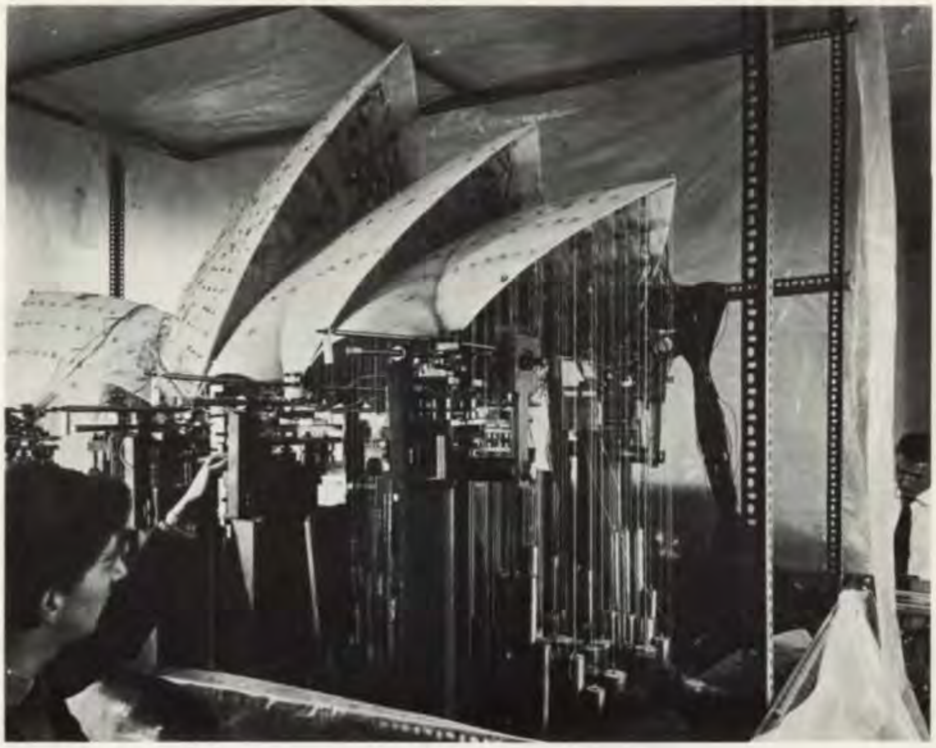


Fig. 19
The structural model test at Southampton University
(Photo: Henk Snoek)

It became evident from the results of the model tests that shear forces and bending moments in the system were higher than had been anticipated. What was more serious, however, was that the model analysis yielded results of load distributions to the foundations which could not be predicted by any analytical technique then known. It would have been possible to cater for these larger-than-foreseen forces and bending moments but at this stage two factors had become quite clear.

Firstly, structural design is an iterative process so that the reshaping required by any weaknesses which were revealed would in itself lead to significant alteration in the distribution of the forces in the system. This is nothing new and is quite predictable but was considerably aggravated by the complexity of the structure and the length of time necessary to investigate each alteration. Secondly, and in many ways more important, the architect who, under the pressure of a premature start on site, had been preoccupied with providing adequate information for the structural design of the base, expressed dissatisfaction with certain specific aspects of the roof and in particular with the louvre walls and the internal appearance.

A detailed reappraisal therefore took place and the possibility of throwing away years of work became a stark reality. This reappraisal essentially consisted of two parallel investigations.

Firstly, the scheme which had been model tested was amended to a steel structure – basically a framework where the concrete skins were used where possible in composite action with steel to resist shear and compression. Moreover, the roof was articulated in a way that made the prediction of column and foundation loads more amenable to calculation (Fig. 18(G)). Secondly, a radically different solution, where the roof surface would be formed by a series of fanlike ribs, was given detailed consideration (Fig. 18(H)). The architect, after his first free undisciplined shapes, was becoming more and more convinced of the beauty, logic and order of a disciplined geometric system, where the geometry was positively expressed rather than subtly implied.

This reappraisal showed that a three-dimensional steel skeletal framework was perfectly feasible but that the concrete surfaces could be used structurally only to a limited extent. The fanlike or folded scheme seemed possible although considerable architectural alterations were necessary. The steel structure, though difficult, would present no great novel problem in execution, but the fanlike concrete proposal, if possible at all, would require the closest possible integration of design and construction.

Faced with the choice, the architect had no doubt what he wanted – the folded solution. He felt that the integrity of a concrete structure left in its natural state was in keeping with the ideals of his concept for the scheme. This forms the basis of the final scheme which was built.

Final shape and description of roof structure

The roof structure covers the two main halls and the restaurant. There are three main elements forming each roof structure (Figs. 20(a), (b) and (c) – main shells (A1, 2, 3, etc.), side shells (D5, 6, 7, etc.), and louvre shells (C9 and 10). The word 'louvre' has found a permanent place in the Opera House Roof terminology from the 'louvre wall' which it replaced. Each of these shells is made up of two half-shells, symmetrical about the central axis of the hall. Each half is a reflection of the other, mirrored about the vertical plane of the hall axis.

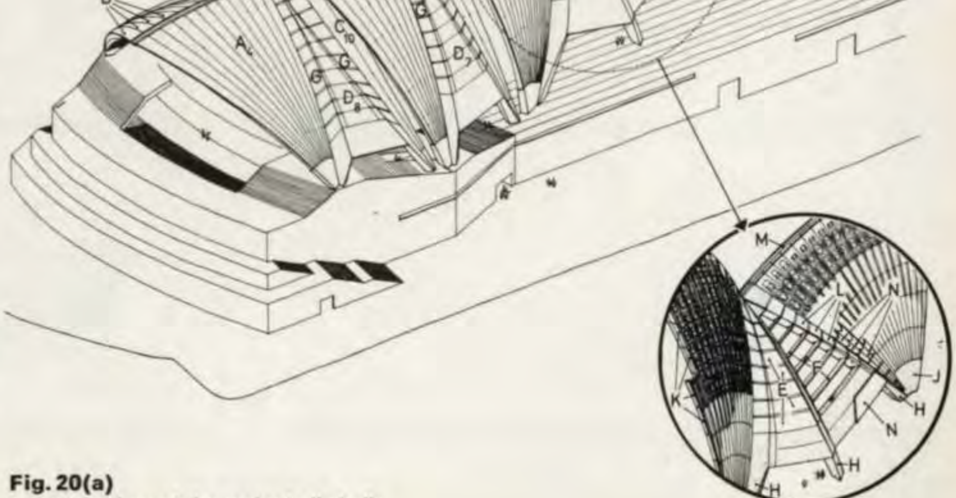


Fig. 20(a)
Isometric view of the major hall shells

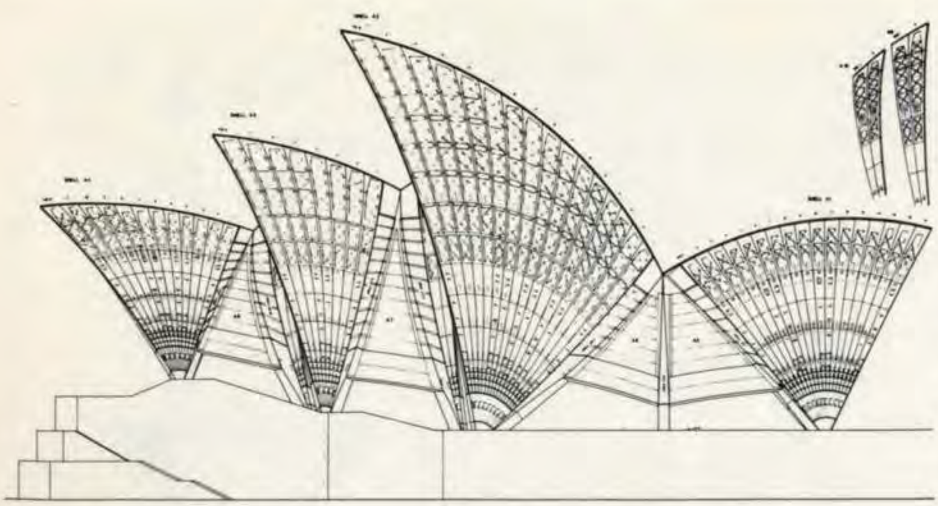


Fig. 20(b)
West elevation of the major hall superstructure

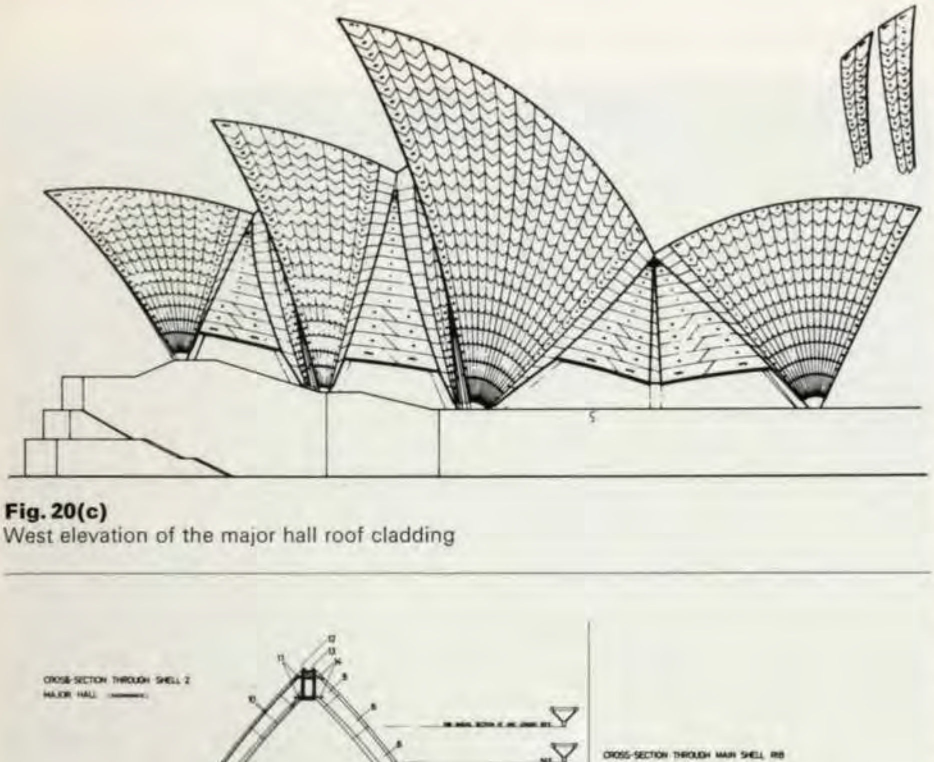


Fig. 20(c)
West elevation of the major hall roof cladding

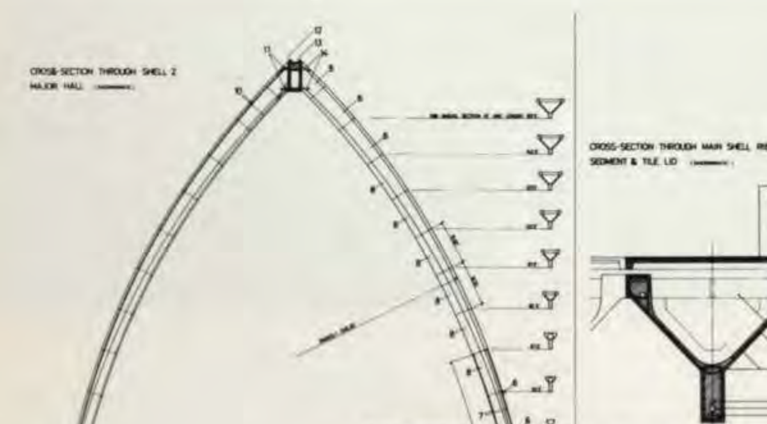


Fig. 21
Main rib segments and cross-sections

The following leading dimensions will give some idea of the scale of the roof structure. The height to the top of the largest shell from its springing point is 54.6 m. The overall length of the larger hall (measured between the tips of the end shells) is 121 m. The overall widths of the main shells range from 22 m to 57 m. The longest rib is made from 12 standard segments and one top segment in each half; the arc length, measured along its centre-line from springing-point to ridge, is 64 m.

Each main shell (A) appears in elevation as a curvilinear triangle standing on one vertex. Each triangle, which forms the outer surface of the shell, is a portion of sphere whose radius is 75 m. On two sides this half-shell or spherical triangle is bounded by great circles and on the third side (the ridge) by a small circle. This small circle is parallel to and 0.6 m from the hall axis plane. The small circle is repeated for the other half of the shell, the cylindrical surface joining the two halves forming the ridge of each pair of main shells. The two great circles converge towards the springing point and meet some way below in a point equivalent to a pole of the sphere. Cross-sections through the roof are still ogival arches and though there are geometric differences from previous schemes, the principles remain the same. The essence is that each main shell has its external surface described as parts of a sphere.

Each half main shell consists of a series of concrete ribs. The centre-line of each rib is a great circle of the sphere. Centre-lines are equally spaced (3.65° apart) throughout each main shell, each centre-line passing through the pole of the sphere. In this way ribs radiate from the podium and they become wider up the shell, successive ribs becoming longer or shorter as the case may be.

The cross-section of each rib varies by smooth surfaces from a solid T at the pedestal to a solid Y and then to an open Y at its upper reaches, the principle being one of a hollow concrete tube, but near the pedestal the ribs are too thin to be hollow (Fig. 21). The ribs end in a vertical plane and a 76 mm joint is left between this plane and the ridge beam to allow for tolerances during construction. The soffit of each rib is formed by a circular cylindrical surface. The centre of the circle forming the cylinder is eccentric to the great circle of the sphere forming the centre line of the rib in such a way as to increase the depth of the rib from pedestal to ridge.

The ribs coalesce into a reinforced concrete pedestal (J) which provides a common springing point for all precast concrete sections. The arc length from the pole to this springing point is 6.9 m for all shells except those over the restaurant where it is reduced to 5.6 m. Although all ribs have different overall lengths, they all have similar cross-sections at equal distance from the pole of the sphere. This principle, namely fabricating similar elements forming part of a spherical geometry, has made it possible to conceive a sensible construction process embodying the maximum use of repetitive elements.

From the top of the reinforced concrete pedestal, each rib is made from segments 4.6 m long. The chosen size and weight of precast segments resulted from a comprehensive investigation into cranage and construction methods. All types of lifting equipment were studied. It became clear that a tower crane would provide the best solution to the problem, provided one of adequate capacity could be found. A crane capable of lifting 10,160 kg 30.5 m from the mast head was considered to be the minimum capacity which could handle precast units of sensible dimensions. 10,160 kg is the approximate weight of a typical segment. A modified Babcock-Weitz crane, G 280 B, fulfilled this specification. Three such cranes were used to erect the precast concrete roof and cladding elements. All precast rib segments for main shells were, therefore, standardized on 4.6 m lengths except the top ones which were cut off by the vertical plane parallel to the hall axis. Consequently, the end details of the top segments are non-standard (Figs. 22 and 23). Precast elements of a rib are stressed together. Anchorages for prestressing cables in the pedestals and the bottom segments of the ribs were architecturally expressed and standardized (Fig. 24). Anchorages in the top segments presented some detailing problems illustrated in Fig. 22.

The ridge is created by fitting a hollow box beam of constant cross-section between the vertical planes of the half shells (Fig. 25). This ridge beam is itself made up of segments, one segment per rib, bolted temporarily and finally stressed together. Each segment forms the keystone of an ogival arch formed by two half ribs and is subsequently stressed to its neighbouring segment to form a hollow circular concrete beam. It should be noted that each half of the ogival arch is in a different plane inclined to, but meeting in, the vertical plane passing through the hall axis.

The louvre shells (C) are identical in principle to the main shells. They are composed of only two ribs and partially fill the space between main shells created by the omission of the original louvre walls. They also form the lower sills for the bronze cladding which fills the remaining gap between the shells.

Side shells (D) are spherical triangles which link main and louvre shells. The geometrical derivation is shown in Fig. 26. Here again each shell forms part of the surface of the same sphere. The meridian passes through the vertex of each triangle forming the side shell. The boundaries of the side shell are small circles. Great circles intersect this meridian at 2.28 m intervals. Although each spherical triangle forming a side-shell is different, this provides a common geometry for setting out and hence fabricating the elements of the side-shell structure. The space between side shell and main or louvre shell is filled by a warped surface described by two points which move up each circle at the same speed and are joined by straight lines.

The arrangement of the side shell structure is as follows: arches (G) form the boundary members whose upper surfaces are the warped surfaces just described. These arches form the primary supporting system of the entire structure. The arch is a hollow concrete tube of continually varying cross-section (Fig. 27). The part facing the main shell contains half a main shell rib section, the part facing the side shell is bounded by a plane surface, the arch boundary plane. Each arch is made of an in situ springing section (H) and of precast concrete segments in a similar way to a main shell rib. In this case, however, the segments are only 2.28 m long due to their higher unit weight. Segments were tensioned together by bolts during erection and were subsequently stressed. Precast beams (F) span between arches. These beams, identical in cross-section and curvature, are of varying lengths. Their ends are cut off at different angles and at different distances from the side-shell meridian. The beams are spaced at 2.28 m centres, parallel to the bottom boundary of the shell and their ends coincide with the joints between the precast units of the arches. The lower boundary of the side shell has a box beam which forms part of the in situ section (H). The surface of the side shell is formed by thin (63.5 mm thick) precast concrete slabs which span between the precast beams. These slabs, which vary in length, were cast as cylindrical surfaces from a single mould which exploited the geometric similarity previously described. The deviation of the cylinder from the sphere is of no practical significance.

Roof cladding

The architect decided to cover the external surface of the roof structure with ceramic tiles. In collaboration with the supplier he developed two tile types – one glazed, one matt, but both off-white. Considerable experience had been gained in the use of similar types of tiles on precast concrete cladding for buildings – mainly in Sweden. Most of this experience related to adhesion between tile and backing concrete and in particular to the optimum size of panel which could safely be constructed without creating problems as a result of temperature variations.

It was decided to clad the structure with precast concrete panels covered with these tiles. The main shell 'tile panels' followed the sphere, each panel being chevron-shaped and 2.3 m long (Fig. 28). The objective was to express the anatomy of the roof structure and the joints between the panels coincided with the joints of the main shell ribs. Each panel is 44.5 mm thick overall and is stiffened with 152 mm deep reinforced concrete ribs. It is heavily reinforced with three layers of galvanized steel mesh. Approximately one million tiles were organized on to the roof surfaces. The typical tile is 120 mm square and non-typical edge tiles were detailed and generally pre-cut in the factory. The spherical geometry permitted a great degree of repetition. In the lower reaches of the shell identical tile panels were repeated as often as 280 times.

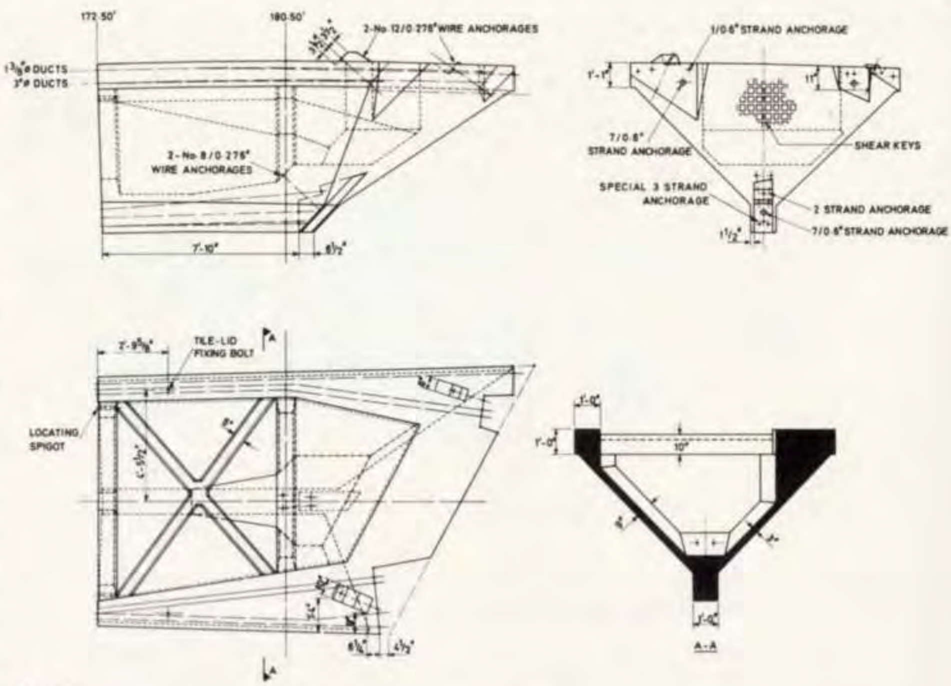


Fig. 22
Top rib segment. Shell A2. Rib 12 (simplified drawing)

Fig. 23
A top segment of a rib
(Photo: Max Dupain)

Fig. 24
Shell A2 pedestal and lower segments showing anchorage pockets
(Photo: David Dowrick)

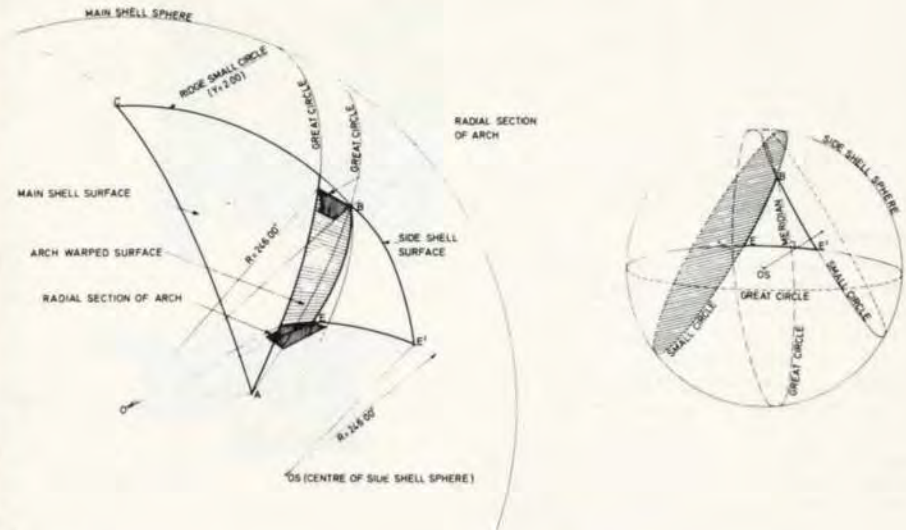


Fig. 26
Geometrical relationship
between main shell
and side shell structures

Fig. 25
A ridge segment going into position
(Photo: Max Dupain)

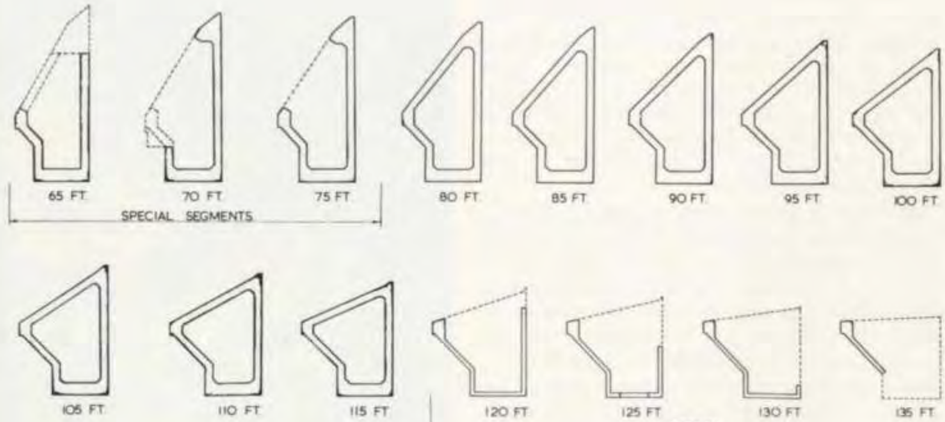


Fig. 27
Radial cross-sections of a precast side-shell arch

Side-shell tile panels were also designed to express the side shell structure. Their construction is similar to those of the main shell. One mould produced all the panels. Warped surface tile panels were all special, 57.2 mm thick unstiffened reinforced concrete and fabricated from one adjustable mould.

The maximum size of panel was limited to simplify handling as well as to control temperature effects. It does not exceed 19.5 m². Special non-ferrous fixings were devised to provide attachments which allowed temperature movements, yet provided adequate resistance to suctions of up to 3832 N/m². The design of

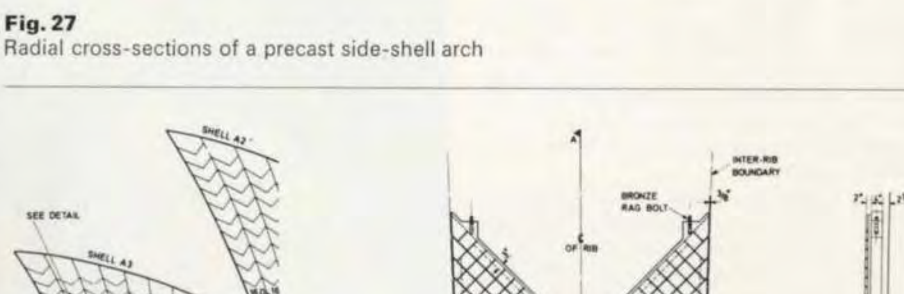


Fig. 28
Tile-lid layout and typical detail

brackets, bolts and other gadgets was further complicated by the curvature of the roof which causes substantial variations in the direction of load application. The attachments have sufficient tolerances to make adjustment possible so as to create the continuous surfaces the architect specified. Fig. 29 indicates some of the brackets, bolts and washers. The materials used were various bronze alloys, notably aluminium bronze to BS1400 for the brackets, and phosphor bronze to BS369 (hard) for bolts and nuts. Ultimate load tests were carried out on some of these components. The maximum weight of the tile panels is 2540 kg.

Construction

In addition to the prestressed concrete ties between shells and the reinforced concrete pedestals, the roof structure consists of over 2400 precast units and more than 4000 tile panels. All precast members were manufactured in a factory established on site (Fig. 30). For the structural components, formwork was generally steel framed with a plywood lining. The plywood was treated with several layers of fibreglass bonded polyester resin to give a smooth and precise finish. All ribs and arch members were manufactured with matching surfaces suitable for a thin 0.8 mm–3 mm maximum epoxy resin joint. Tile panels were manufactured in concrete moulds on which aluminium strips set out the tile positions and provided the recessed mortar joint the architect had specified (Fig. 31). Differential shrinkage between the tiles and the concrete backing was largely controlled by the stiffening ribs but the curvature of the solid warped surface panels had to be specially adjusted.

After the pedestals and the prestressed ties had been constructed, the arch structure was erected. It was assembled on fabricated structural steel centering. The segments were stressed together, in situ joints made at pedestal and crown and the precast beams were erected and jointed. The primary structure was, therefore, completed before erection of the shell proper began.

Erection was carried out by means of the three tower cranes. The capacity of each crane was 300 tonne metres. Two cranes, travelling on special steel bridges, erected roof segments in the northern part of the structure while the third erected most of the segments in the southern section, including the restaurant.

Main shell ribs were generally assembled by supporting the precast segments with needles between a specially designed steel erection arch on the one side and the already completed structure on the other (Figs. 32 and 33). Four such arches were constructed for the erection programme. The erection arch had a circular profile and a telescoping mechanism for adjustment to suit longer and shorter ribs. It was moved from one rib position to the next by hydraulic rams.

Temporary stressing during erection ensured that the joints between segments were under compression during setting of the epoxy resin. The system was designed to ensure that the deflections of the supporting falsework could not induce excessive tensile stresses in the structure under temporary conditions. Each half-rib was then permanently stressed, joined to the pedestal and the two half-ribs were in turn joined by stressing across the ridge. Some details of stressing equipment are given later. To allow for movements between rib and rib and between rib and arch during construction, ribs were erected on low-friction bearings. Loads on arches were controlled by flatjacks which were progressively adjusted and removed as construction progressed.

Due to deformations, mainly differential, of the comparatively flexible erection arch and the already completed structure, the needles could be adjusted so that errors in alignment could be corrected as erection proceeded. A comprehensive computerized surveying system was established to allow these adjustments to be

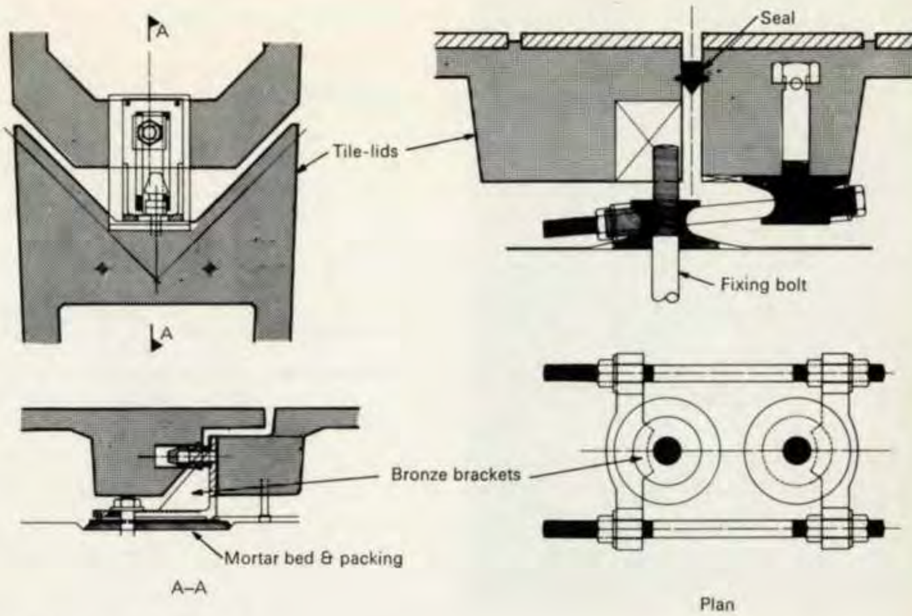


Fig. 29
Two types of tile-lid fixings

Fig. 30
The precasting yard and the site
(Photo: Max Dupain)

made without impeding the progress of the work. The specified precision of the completed structure was such as to make the whole question of tolerances, and hence the survey control, of primary importance.

During construction, tensile forces between ribs were resisted by bolts. Although the structure was prestressed laterally after erection, some bolts were considered to be desirable to provide adequate load factors under ultimate conditions. They were, therefore, made of highly corrosion-resistant material. At first, only Titanium *314A* appeared to have the necessary proven fatigue resistance. However, subsequent tests showed *K-Monel D*, a copper nickel alloy, to have equal or better ultimate and fatigue resistance. Because of its higher unit cost Titanium was used only until this was established. The bolts, which are up to 2.5 m long, have rocker assemblies at their ends to allow for movement, particularly during construction.

Lateral stressing took place at various stages during construction after the nominal 25.4 mm gap between ribs had been caulked with dry mortar. Shear keys were formed between the upper radial faces of the ribs to facilitate transference of inter-rib shears.

The main shell structure was made watertight by the application of a sprayed PVA membrane on all precast concrete members before erection. Lead flashing bridges the joint between ribs. The side-shell structure was covered by sprayed white PVC reinforced with fibreglass. Tile lids were erected on bolts which had been accurately surveyed so that tile lid attachments

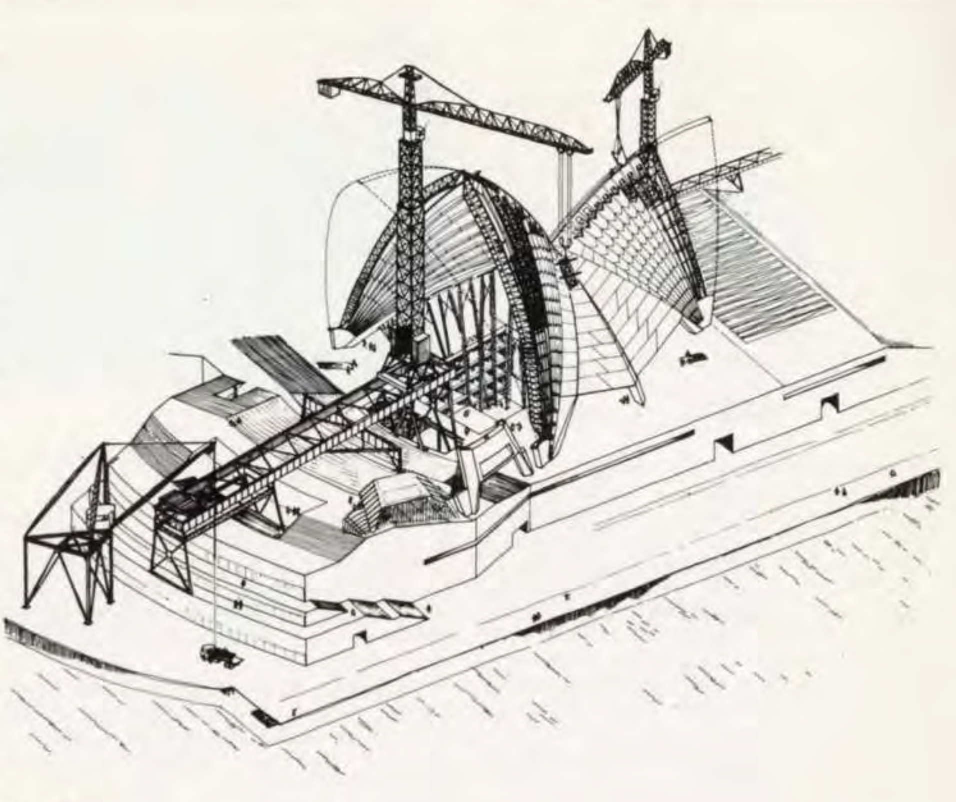


Fig. 32
Isometric view of roof erection

Fig. 31
A tile-lid mould with aluminium strips (Photo: Courtesy *Structural Engineer*)

Fig. 33
Erection of a segment (Photo: Max Dupain)

were chosen to suit the tolerances required. The underside of the tile lids were sprayed with a foamed polyurethane for thermal insulation. Joints between tile lids are waterproofed with Monolastomeric on a plastic backing strip. There are, therefore, two barriers to water penetration.

Concrete mixes for the roof structure varied for different elements. All prestressed concrete work was fabricated from mixes whose minimum specified cube crushing strengths exceeded either 41 N/mm² or 52 N/mm². Careful statistical controls were kept on site and average strengths were well in excess of those specified. Approximately 13,000 m³ of concrete were used for the roof structure.

Prestressing tendons and equipment

The bulk of the prestressing tendons for the roof was made up from 15.2 mm diameter strand, the remainder from 7 mm diameter wires. Because of the high prestress losses, friction in particular, the strand was in the stabilized condition to minimize relaxation. The tendons made from wires were used for most of the 2.44 m long continuity cables at the ridge. It was easier to overcome the high prestress losses due to draw-in at the anchorage grip with wires rather than strands. In the ribs and arches, prestressing was effected with 15.2 mm diameter strands in groups of 1, 3, 4, 5, 6 or 7 depending on the force required. To standardize ducts three sizes were used, 35 mm, 51 mm and 76 mm diameter all formed with Ductube. Anchorages were the flat-plate types supplied by CCL. No spirals or helices were used because of the congestion of bursting reinforcement in what were frequently complex anchorage zones. For the temporary stressing and destressing during rib erection, a 'flying-wedge' system of anchorage was used. This was a special adaptation of a principle patented by PSC Equipment Ltd. It involved both male and female wedges in the anchor block to facilitate destressing (Fig. 34). The jack for the temporary stressing was a special double-piston model made by CCL (Canada) (Fig. 35). It allowed the strand to be threaded into the jack sideways. This was virtually essential for the 'flying-wedge' system as it involved frequent stressing and restressing of several running lengths of strand. The jacks for

Fig. 34
Rib during erection. Grips for temporary anchorage being fitted (Photo: Max Dupain)

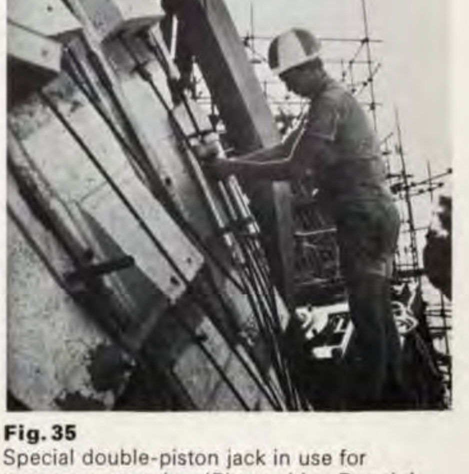


Fig. 35
Special double-piston jack in use for temporary stressing (Photo: Max Dupain)

the permanent prestressing forces were standard types which were simpler to use and more reliable.

Measurement of prestressing load was done directly from the jack pressure gauges which were recalibrated at least once a week. A special pressure gauge was used where unusual accuracy was required, for example when very high loads were being used to overcome exceptional friction losses.

258 km of 15.2 mm diameter strand, requiring about 24,000 stressing operations, was used. The double-piston jack performed 10,000 of these operations in the temporary erection procedure and 1000 for the permanent stressing. The *Titan* jack did the remainder of the permanent stressing. At the ridge, continuity cables of 7 mm wires required 6000 stressing operations.

Structural analysis of the roof

In its simplest structural form each main shell was regarded as made from a number of individual tied arches (i.e. the ribs). The two legs of each arch lie in different planes, neither of which is vertical. The plane containing one leg of an arch is a mirror image of the plane of the other leg. To make structural sense of an arch of this form it must be supported at the ridge, and a simple tension-compression support tangential to the ridge circle was used (Fig. 36). In addition each rib was supported off the previous rib by tension-compression connections at discreet points between the pedestal and the ridge in order to reduce the torsional (M_t) and lateral bending (M_r) moments. The direct forces in the ridge and in the rib connections built up algebraically from zero at the free edge to tensions or compressions at the side shells. They were transmitted to the side-shell complex through flatjacks where it is necessary to monitor and control the loads or through sliding bearings. The centroid of the load on each main shell lies close to a vertical plane through the pedestals so that the net overturning effect on the side shells was relatively small. The system described was the simply-connected structure and represents the condition of the main shells immediately after erection.

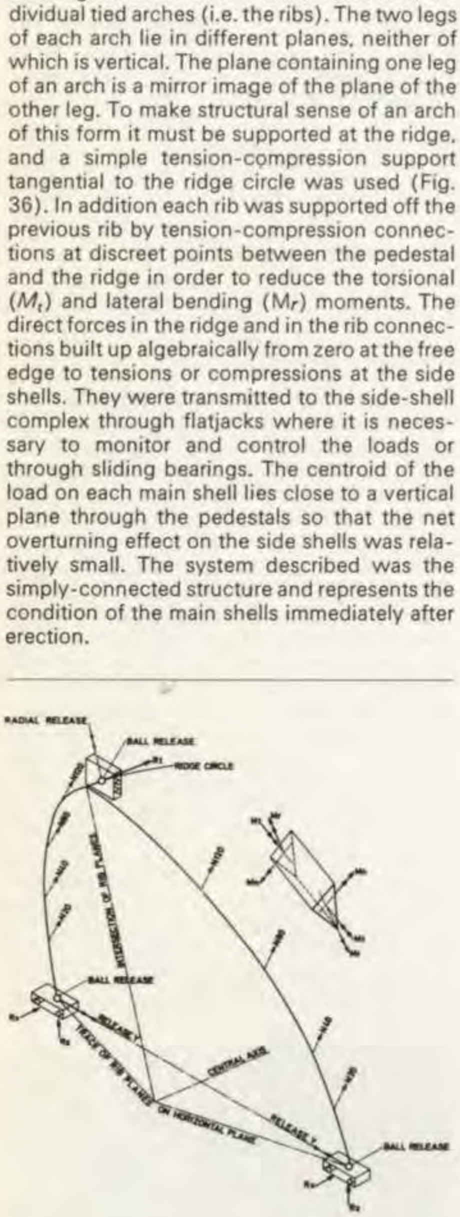


Fig. 36
Typical rib release system for the structural analysis

Finally, the 25.4 mm gaps between the rib flanges were filled with mortar and the lateral cables stressed. This could have been progressive provided that it was done so far behind the erection that relative movements of adjacent ribs were negligible. When completed, this produced what is known as the laterally-stressed structure which behaves as an anisotropic shell, supported at the pedestal, continuous over the ridge beam and simply-supported tangentially at the side shell. (For some shells the flatjacks have been removed and the only support given by the side shell is to the ridge).

Fig. 37
Plan view of wind model test with tufts showing wind directions (Photo: Ove Arup & Partners)

The side shell structure acts essentially as a tetrapod tied at the feet. The arches are the legs and the side shell beams and box beams form strut-ties (with end moment-connections) between pairs of legs.

Loading conditions

Seven types of loading were considered:

(a) The dead weight of the shells
This includes the self-weight of the structure, the weight of the tile lids and a small superimposed load of 49 kg/m² which was allowed for maintenance loadings, air-conditioning ducts, etc. on the main shells while for the side-shells the superimposed load was increased to 147 kg/m².

(b) Wind wall loads
The window walls which close off the north and south ends of each hall are substantial structures. Mullions for the wall below shell A4, for example, span a maximum height of 22.2 m between podium level and the rib which supports them. Because of the shape of the mullions (they are cranked on side elevation) a substantial part of the weight of the window is hung from the rib, and wind pressures and suctions on the window impose vertical as well as horizontal loads on the rib. Smaller loads are applied to ribs on the other shells from the bronze-sheeted walls which close off the gap between each louvre shell and the shell above.

(c) Auditorium ceiling
The average weight of the auditorium ceiling is slightly greater than 488 kg/m² on plan. This high value is necessary to give adequate sound insulation. The construction is arranged so that the ceiling is not supported by the main shells but by the side-shell arches or directly by the substructure.

(d) Wind
Comprehensive records of wind speeds, directions or gust durations were not available for Sydney.

However, there were several recorded gusts of 145–153 km/h and in 1876 a speed of 185 km/h may have been exceeded. Tornados off the Queensland coast occasionally produce even higher wind speeds and the possibility of one straying south to the Sydney area had to be considered.

One limit state considered was that at which there was just no tension in the prestressed sections and permissible working stresses were not exceeded, with a windspeed of 161 km/h. The critical ribs are those outside the window walls where, for some wind directions, the wind forces on the underside act in the same sense as those on the top. These are also the critical ribs for the limit state of collapse and failure was predicted for a wind speed of 274 km/h. However, this would occur only in the flexibly connected structure. Once the lateral stressing is effective then the critical wind speeds are very much higher since the wind pressures are now distributed more by membrane action. The performance of the structure under wind was, therefore, considered satisfactory particularly as the limit states would only be attained with a restricted range of wind directions.

Estimates of the wind-pressure distribution over the shells for various wind directions were obtained from wind-tunnel tests at Southampton University and at the National Physical Laboratory. The model used was of solid wood representing the roof of the major hall and part of the base to a 1:100 scale (Fig. 37). A number of subsurface ducts was formed in each shell, parallel to the ridge, and each duct was connected at its lower end to a manometer. By opening these ducts to the air at points successively closer to the lower end, pressure readings for many points could be obtained from each manometer. To form the ducts, nylon wires were laid in 3.2 mm diameter grooves cut in the surface of the shell. The grooves were then filled with resin and the wires withdrawn.

The only wind tunnel available at Southampton University had a closed working section 1.22 m square and the model occupied a large proportion of the stream area, particularly when at right angles to the wind. It was realized that the high blockage factor would lead to an over-estimate of the leeward suctions; so when an opportunity of using the NPL 2.74 m × 2.13 m open jet tunnel occurred, further measurements on the more critical area (i.e. Shell 2) were made. The second test was carried out by R. E. Whitbread and Miss M. A. Packer of the Aerodynamics Division of the NPL[5]. Fig. 38 shows pressure contours obtained for Shell A2 with the wind blowing from N 120° E (one of the critical directions). A typical difference between the two tests was that for wind from due east ($\theta = 90°$), where the Southampton tests indicated a range of suction on the leeward face of 0.29 to 0.48 kN/m² (for 161 km/h wind) whereas the NPL tests gave a range of 0.09 to 0.14 kN/m². The windward pressure distribution was virtually unchanged. Maximum suctions measured on Shells 2 and 4 were respectively 1.85 and 3.5 times the stagnation pressure. These occurred very locally on the leeward side of the ridge but had to be allowed for in the design of the tile lids and fixings. The factor 3.5 was found in the Southampton tests only and may be slightly high.

To examine the airflow in the NPL tests tufts were attached to the surface of the model. Observation of these tufts indicated that flow separation was confined to the sharp edges (the ridges) of the model. It is reasonable to assume that the roof is a bluff body, that pressures are independent of Reynolds Number and that the tests results give pressure distributions which were adequate for design purposes. In assessing the limit states account had to be taken of a wind velocity gradient with height and of the presence of the other hall, neither of which were represented in the tests.

(e) Temperature

Four temperature conditions were considered in the analysis.

1 A uniform temperature change of the whole roof relative to the supporting structure.

2 A uniform temperature change of the east half of the roof relative to the west half.

3 A radial temperature gradient, i.e. through the thickness of the roof.

4 A temperature gradient along the roof.

For the side-shell analysis, only the first of these was found to be important. In the permanent state (with the insulated tile lids in position) an assumed temperature difference of 5.5°C (10°F) gave bending moments at the feet of the arches roughly 15 per cent of the total. Severer conditions had to be allowed for in the temporary state before the tile lids were placed.

For the main shells, a uniform temperature change was less important because the ribs were more flexible radially than the side shell arches. However, temperature gradients along the shells gave very high bending moments at the top of some ribs (requiring additional continuity cables at the ridge) and high inter-rib shears for the laterally stressed structure. These temperature gradients occur because the shells oversail the window or louvre walls. In Shell A4, for example, the window wall will be glass rising from podium level to the soffit of the fourth rib from the outside. The summer design temperature distribution was taken as 27°C (80°F) for internal ribs and 43°C (110°F) for the external ribs. Ribs at, or next to, the external walls were given intermediate temperatures and were subjected to a gradient across their width. The end rib could receive some energy direct from the sun and was calculated to have temperatures varying from 46°C (115°F) to 55°C (130°F) across its width in its hottest condition.

For the smaller shells, like A1, these

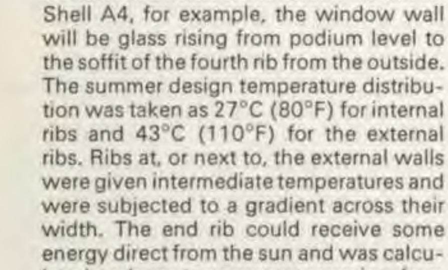


Fig. 38
Typical wind tunnel test result. Pressure distribution on Shell A2

temperature differentials could be reduced because of the higher ratio of mass/surface area for the ribs which means that their average temperature never reaches that of the ambient air. This was convenient since these shells were stiffer and more sensitive to temperature differentials. Winter conditions were taken to be similar but of course reversed. Another less severe temperature gradient that had to be considered in the analysis of every rib could occur temporarily while the tile lids were being erected. The unclad ribs would receive solar energy directly and become hotter than those protected by the tile lids at roughly air temperature.

(f) Creep

Creep is usually important only for the deformations of a structure and its effect on concrete stresses is normally beneficial. This may not be true where the structural system is changed after the application of a permanent load. Two such changes had to be considered in the roof analysis.

The first occurred whenever a new rib was completed and stressed across the ridge. Because of the difference of loading history between adjacent ribs, their creep deformations due to dead load and prestress are proceeding at different rates. Because adjacent ribs are almost identical, the ultimate incompatibility between them will be numerically equal to the creep deformation that has already occurred in the earlier rib when the later rib is completed. The shear induced at rib interfaces due to creep was reduced along their length by tension-compression connections during erection. Creep stresses were, therefore, unimportant for the normal erection procedure with the ribs being built continuously. The construction of Shells 2 and 3, however, had to stop four ribs from the end while the louvre shells (which lie vertically below the end part of the shell) were erected. The consequent two-month delay between adjacent ribs meant that the stresses caused by creep under prestress were significant. They were, in fact, taken as equivalent to those that would be caused by a temperature drop in the last four ribs of 10°C (18°F) relative to the earlier ribs.

The second crucial change occurs when the inter-rib concrete is placed and the lateral cables stressed. If f_f and f_s are the stresses caused by deadweight acting on the flexibly connected structure and the laterally stressed structure respectively, then the stresses will change slowly from f_f at the time of lateral stressing to a final value intermediate between f_f and f_s. This final value is $f_f + \frac{(r-1)}{r}(f_f - f_s)$ where r is the ratio of a final deflection (including creep) to the instantaneous deflection (assuming the structure does not change). The value taken for r was 2.8, a rather conservative value since the instantaneous deflection here means the deflection at the time of lateral stressing so that it will already include a substantial amount of creep. Of course, f_f is usually numerically larger than f_s, so that the stresses decrease with time; but there were important stresses such as inter-rib shears for which this was untrue.

(g) Stresses due to the erection procedure

To check the additional stresses occurring during erection was, of course, routine, although the precautions necessary with the longer ribs are worth mentioning. These ribs had to be supported on the steel erection arch through hydraulic jacks, otherwise the deflection of the steel arch would have opened up the joints in the partially built rib. Charts were prepared to show, graphically, the allowable combinations of these jack loads so as not to produce tension in either the inner or outer face of the rib. Similarly, restrictions had to be calculated on the number of ribs for which the construction of Shell 2 could be ahead of Shell 1 before the unbalanced effect overstressed the side shells.

Of more interest are the erection stresses that were built in and contributed to the final stresses. Parasitic stresses due to prestressing are an obvious and orthodox example. These were avoided almost completely on the main shell ribs by arranging for the prestress force to be as nearly centroidal as was possible in a varying section and by providing sliding bearings between ribs. This was confirmed by observation of the jack loads (on the erection arch) which did not change appreciably during the application of the final prestress force. In the side-shell arches the stress was less uniform, but calculations showed that the stiffness of the scaffold support was low enough to make the effect negligible.

The method of support of a completed half-rib at the time when continuity was made across the ridge could also build in stresses. For, suppose a half rib is stressed and, at the time of making the ridge joint, supported only on needles at the top and bottom; then the top of the half-rib will have rotated due to the self-weight of the rib spanning between the needles. The built-in stresses can then be calculated from this rotation applied as a forced rotation to the complete redundant structure. Of course, in this instance, the algebraic addition of these built-in stresses to the stresses calculated for the complete redundant structure will give total stresses equal to those obtained assuming the ribs are hinged at the ridge for their self-weight loading. When the rib was supported by intermediate needles it was found possible to choose the needle loads so as to give zero rotation.

Method of analysis

The method of analysis relied very heavily on the use of electronic computers. It is difficult to visualize how the necessary calculations could have been made without them. It must be remembered that this was one of the first large-scale applications of electronic computers to a building structure and was made at a time when the capacity and speed of the machines, the number of available programs and the sophistication of the languages, were very much less than they are now. Perhaps more important, there was not, at the beginning anyway, a complete understanding among practical engineers of which analytical approaches were best suited to the machines' capabilities. Therefore, it is certain that if the calculations had to be repeated, the details of the analytical process would be vastly different although it is possible that the main lines of attack would be substantially the same. While the analysis will not be described in great detail here, the experience gained in the use of computers provided the basis for much subsequent development.

Each side-shell complex was analysed as a complete structural unit, independent of the main shells. Allowance was of course made for the thrusts from the main shells (which varied according to the stage of erection reached (Fig. 39). In early studies made to understand the behaviour of the side shells and determine suitable member sizes, the shells were represented by heavily simplified structures with seven redundants. These were analysed initially by hand and later by a specially written flexibility matrix program.

For the final analysis each side-shell structure was simulated by a rigid three-dimensional space framework of straight prismatic members. This was analysed on a Ferranti Pegasus Computer using a general space framework program based on the stiffness method. This framework program had been written in machine-code by Dr A. Baker, one of the engineers working on the project[6]. The program, as originally written, was limited to structures with 18 or fewer joints, but was later extended by a partitioning device to cope with larger structures. As a typical example, side-shell complex A7 (or the western half which was by symmetry, all that was necessary) was simulated by a 35-joint framework. The analysis of this, with five load cases, took 55 minutes on the Pegasus computer.

Additional time was taken on the Ferranti Sirius Computer; initially to calculate section properties, and finally to combine the calculated bending moments and to transform them to principal axes. This analysis was later checked by Dr Renton's space framework program on the Cambridge Edsac II machine.

All main shells had to be analysed fully in both the simply-connected and the laterally-stressed conditions. For the simply-connected structure, the analysis was by flexibility methods using a series of programs specially written by Peter Rice, who had been working on the earlier schemes and had gained considerable experience with computers in the calculations. The basic unit in this analysis was the individual rib made statically determinate by the releases shown in Fig. 36. The redundants for a complete shell were then the redundants corresponding to these releases in every rib plus interaction forces between adjacent ribs (or for the lowest rib the reaction from the side-shell). After early investigations these inter-rib forces were limited to forces normal to the great circle plane bisecting the two rib planes and for ease of programming they were positioned at arc lengths from the pole that were multiples of 6 m. Programs were written, usually using the Pegasus Matrix Interpretive Scheme, to perform each of the following steps in the analysis:

1 Given the basic sphere and position of the first rib, to develop the geometry of every rib at 3 m intervals
2 Given the loading, to calculate the reactions in each released rib
3 From (1) and (2) to calculate the free moments for each rib
4 To calculate the bending moments due to unit value of each redundant
5 To calculate the flexibility matrix and the corresponding load terms. This was done by Simpson's rule using a 3 m interval taking into account the two bending moments and a twisting moment (i.e. M_n, M_r and M_t in Fig. 36).
6 To select from this flexibility matrix a reduced flexibility matrix corresponding only to those redundants required for the particular structure
7 To invert the flexibility matrix and solve for the redundants
8 To calculate and print the final total moments (M_n, M_r and M_t).

Shells 3 and 4 could be analysed as complete units but Shells 1 and 2 had to be dealt with in two halves. A Pegasus Mk 1 machine, without magnetic tape backing store, was used for the computations. Steps 1 to 5 took a total of about 1½ hours machine time when there were 150 redundants. The largest flexibility matrix inverted was of the 98th order and for this a further 1½ hours were needed for steps 6 to 8. Figs. 40 and 41 show some typical BM diagrams.

A flexibility, rather than a stiffness, method of analysis was used, principally because there was not, at that time, a large enough stiffness program readily accessible. But the decision could be clearly justified on other grounds. The most important of these was the facility,

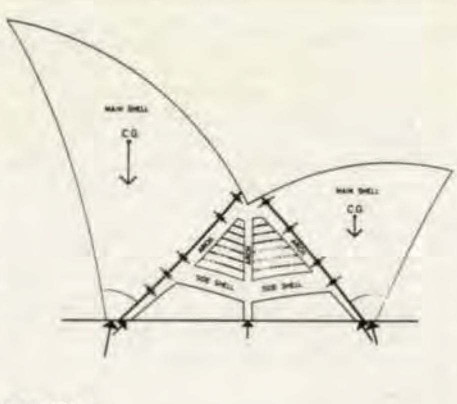


Fig. 39
Articulated structural unit

contained in step 6, for varying the structure. Thus steps 1 to 5 were made only once for each shell (or half-shell) and included all possible redundants, i.e. the inter-rib forces including tangential and radial shears at all permissible points. Then, for each structural scheme investigated, the non-existent redundants were eliminated and only steps 6 to 8 were required. This was convenient both for the early design (when the effect of inter-rib shears was being studied and the optimum positions of the lateral connections determined) and for the final analysis (when the behaviour of the shells during erection with varying numbers of ribs in position had to be investigated). In addition, the flexibility method makes the best use of the repetitive geometry of the ribs and is more efficient for members of circular profile and continuously varying section properties.

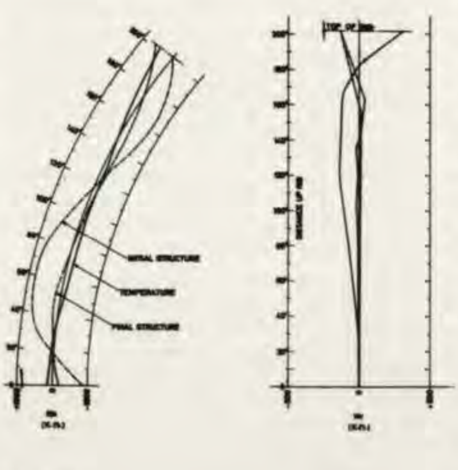


Fig. 40
Bending moment diagrams for a long rib (*Mt* not shown). Dead load and temperature conditions only

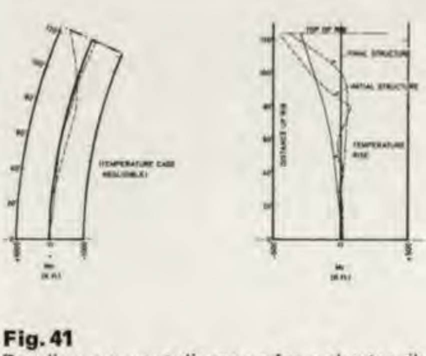


Fig. 41
Bending moment diagrams for a shorter rib (*Mt* not shown). Dead load and temperature conditions only

This analysis had to be refined in certain areas by additional calculations performed either by hand or using subsidiary computer programs. Corrections had to be made for the fact that it was not practicable to place the lateral connections in their exact theoretical position. Also the stiffness values assumed for the lower lengths of the individual ribs (where they coalesced into one pedestal) had to be confirmed. The effect of deflection of the supporting side-shell arches and of elasticity of the lateral connections (non-linear) had to be calculated and this was done by hand, using deflections for suitable unit forces obtained from the main program. The effect was important for the top row of lateral bolts which tended (when they were assumed inextensible) to attract large forces from ridge continuity effects. For temperature and prestress creep loadings, which occurred after the ridge segments were stressed together, the bending stiffness of the ridge beam had to be included. This was done by a program which analysed the ridge as a beam supported on springs. The stiffness matrices of these springs and their fixed-end displacement vectors were found from the main program which was also used to calculate the moments in the ribs from the calculated ridge displacements. The treatment of wind loading was similar but here the effect was largely anti-symmetric and the concern was with sideways movements of the ridge rather than movements in a vertical plane.

For the laterally stressed structure, the large capacity stiffness program developed for the side-shell structure was available. The stiffness method was now more suitable than the flexibility method since not only were the number of redundants very large but the analysis was concerned with the completed structure only. The analysis of the equivalent space frameworks representing the laterally-stressed shells was straightforward but tedious. The largest framework had 136 joints (about 780 equations) and took nearly four hours machine time for five load cases. The calculation and preparation of the input data took approximately three weeks.

Some special problems

The design and construction of the unusual structure threw up many detailed problems, some of great interest. Three items have been selected for a short description. There are several others and it is hoped to publish these at a future date.

Lateral tension connections between ribs

The *K-Monel D* and Titanium *314A* lateral bolts connecting the ribs are in the simply-connected structure subjected to fluctuating tensile stresses due to temperature changes. These fluctuating stresses are very much reduced in the laterally-stressed structure but it was decided that it would be prudent to design these bolts for a fatigue resistance appropriate to a 300-year life in the simply-connected condition. Accelerated corrosion tests were carried out on complete bolt assemblies by Messrs Sandberg, as were fatigue tests on simulated bolt assemblies by Professor Biggs at the University of Cambridge. These fatigue tests gave the Smith Diagrams for 10^5 cycles (Fig. 42) and the *S–N* diagrams for a mean stress of 386 N/mm² (Fig. 43). The curves for *K-Monel* (Short) refer to material which was deliberately given a shorter heat treatment to ease delivery problems; these bolts were used in less highly stressed positions. The bolts were then designed to limit the fluctuating stress to 80 per cent of the value given by the Smith diagram provided that the maximum stress never exceeded 432 N/mm², i.e. one-half of the 0.2 per cent proof stress (the minimum ultimate tensile strengths were 958 N/mm²). It is important to note that there is a safety factor inherent in the selection of 10^5 cycles because the design fluctuating stress (f_s) will not occur every day. In fact if it is assumed that each year for 300 years f_s is

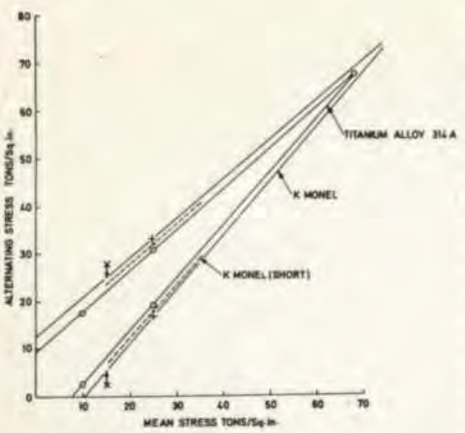


Fig. 42
Lateral bolt fatigue test. Smith diagram

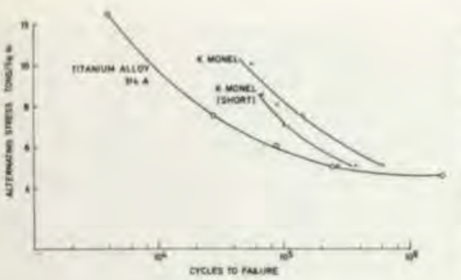


Fig. 43
Lateral bolt fatigue test. *S–N* diagram

reached on 30 days, 1.5 f_s on 20 days, 2 f_s on 10 days, and 0.9, 0.8, 0.7, 0.6 and 0.5 f_s each on 60 days, then by the accumulated damage criterion the effect is less severe than if f_s is reached every day (which is implied by 10^5 cycles). This criterion is that $\sum \frac{n_i}{N_i}$ should not be greater than unity where n_i is the number of cycles for which a stress level f_i is reached and N_i is the allowable number of cycles for that stress level[7]. In fact $\sum \frac{n_i}{N_i} = 0.78$ for *K-Monel* and 0.96 for titanium. It is interesting to note from the *S–N* diagram that although the fatigue properties of titanium are inferior for 10^5 cycles the reverse is true for a number of stress repetitions greater than 10^6.

Epoxy-resin joints

In 1961 feasibility tests were carried out by G. D. Base at the Cement and Concrete Association crushing concrete cubes joined with epoxy resin[8]. In 1963, the contractors for the roof arranged for a series of trials at the University of New South Wales using the selected formulation in order to find the best techniques of surface preparation and resin application. The chosen treatment for the matching surfaces of the segment ends was to grind the surface to just expose the aggregate not more than 10 days before jointing. Immediately before application contact faces were treated by wiping with dewatered absolute alcohol to dry and clean the surfaces. The resin was applied to both faces. On the lower face it was placed and spread with a spatula or grooved scraper and the upper surface was coated using a paint roller.

These methods proved completely successful on site and sound joints were obtained consistently provided a precompression was applied to the joint during the setting period. In 1963–64 Professor Carmichael at the University of New South Wales carried out an extensive programme of tests to determine the effect of various factors on the strength of the joints and to investigate shear creep of the joint. For example, his tests suggested that the strength was reduced by ageing and weathering of the prepared concrete surface and he recommended a reduction of the maximum time interval between grinding and bonding to seven days. He found that creep was negligible with a shear stress of 1.4 N/mm². With a shear stress of 2.8 N/mm² the creep rate could be 5×10^{-6} mm/h and still constant after 90 days. He confirmed the results of earlier workers that the creep strain decreased with increase of joint thickness and was substantially reduced by the application of a compressive stress across the joint (44 per cent reduction for 6.9 N/mm² prestress).

Details of the resin formulation, their strengths and setting times have been published elsewhere[9].

Computer applications

Some indication of the application of electronic computers with respect to the structural analysis has already been given. Extensive use of computers was also made in other ways. So much geometric information for all parts of the structure and its cladding was needed that it could sensibly be provided only by the computer. This geometric information was required for three purposes – for design, for drawing three-dimensional shapes and for setting out on site. The computer helped to minimize the possibility of errors, as well as to save time, and was programmed to give information tabulated either to suit rapid conversion into drawings or to suit being fed to the craftsmen on site, who set out some of the works directly from the computer print-out.

The contractor made use of the computer for the design of the erection arch, statistical control of concrete strengths and job costs; interpretation of the latter sometimes led to design changes in order to achieve economy. Wages were also handled by the computer.

During its erection the roof's shape had to be precisely controlled by constant surveys. By appropriate use of the computer, complex survey computations were carried out very rapidly, allowing savings in time and cost compared with traditional procedures.

Contractual arrangements

The construction of the project has been divided into three stages.

Stage 1 was the structural and civil engineering work for the base of the building. The contract was let to Civil and Civic Contractors Pty Ltd in February 1959. Competitive tenders had been received on the basis of an approximate bill of quantities. The contract was completed in 1963 at a cost of \$5,200,000 (Aust.).

Stage 2 was the structural and civil engineering work associated with the erection of the roof structure and its cladding. It also included parts of the base which could not be completed independently of the roof structure as well as waterproofing basements, structural steelwork for the flytowers and its cladding, and other miscellaneous items. This contract was negotiated with a firm selected by the consulting engineers, M. R. Hornibrook (NSW) Pty Ltd on the basis of cost plus a fixed fee. The contract was completed in 1967 at a total cost of just over \$12,000,000 (Aust.).

The contracts for both Stages 1 and 2 were under the direction of the consulting engineers in accordance with the wishes of the architect who, nevertheless, exercised his discretion with respect to all matters of planning and finishes.

Stage 3 consists of all the work necessary to complete the building. This contract, which will largely consist of a number of major subcontracts, has been negotiated with M. R. Hornibrook who will manage the works for an agreed fee. Although essentially a finishing contract, a considerable amount of structural work still has to be done. Apart from the structure for the glass walls hung from the main shells, there are those which close the side-shell openings. These all have considerable geometric as well as structural difficulties and involve maintenance and corrosion problems. The auditoria ceilings, also part of Stage 3, are quite independent of the roof structure and pose substantial problems acoustically, structurally and constructionally.

After Utzon's resignation a number of major functional changes were made. The decision to use the large hall exclusively as a concert hall, the rearrangement of virtually all the other public accommodation and the provision of greater seating capacities than those proposed by Utzon will all lead to structural alterations and additions inside the halls. Stage 3 is naturally under the direction of the architect.

Conclusion and acknowledgments

Jørn Utzon was the architect who won the original competition and the architect appointed for the project, until his resignation in February 1966.

Hall, Todd and Littlemore are the architects in charge of Stage 3, responsible for the completion of the project.

MacDonald, Wagner and Priddle, Consulting Engineers, of Sydney, were associated with the supervision of Stage 1.

D. C. Gore, BE, MIE(Aust), was the director and project manager for M. R. Hornibrook who, together with his colleagues, built the roof structure. Without his skill and courage what was very difficult may well have become impossible.

The work is being carried out under the general direction of the Minister for Public Works for the Government of New South Wales.

The structural and civil engineering work described in this paper was carried out in the London and Sydney offices of Ove Arup and Partners, Consulting Engineers, and is the product of years of effort by many members of the firm. It would take too long to name them all, but it would be inappropriate not to mention that this unusual structure is a tribute to their skill and ingenuity.

The authors are indebted to J. C. Blanchard and D. J. Dowrick for their assistance in the preparation of this paper.

References

(1) CURTIS, R. E. A vision takes form. A. H. & A. W. Reed, 1967.

(2) BAUME, M. The Sydney Opera House affair. Thos. Nelson Ltd, 1967.

(3) YEOMANS, J. The Other Taj Mahal: what happened at the Sydney Opera House. Longmans, 1968.

(4) ARUP, OVE *and* JENKINS, R. S. The evolution and design of the Concourse at the Sydney Opera House. *ICE Proceedings, 39,* (April) pp.541–565, 1968.

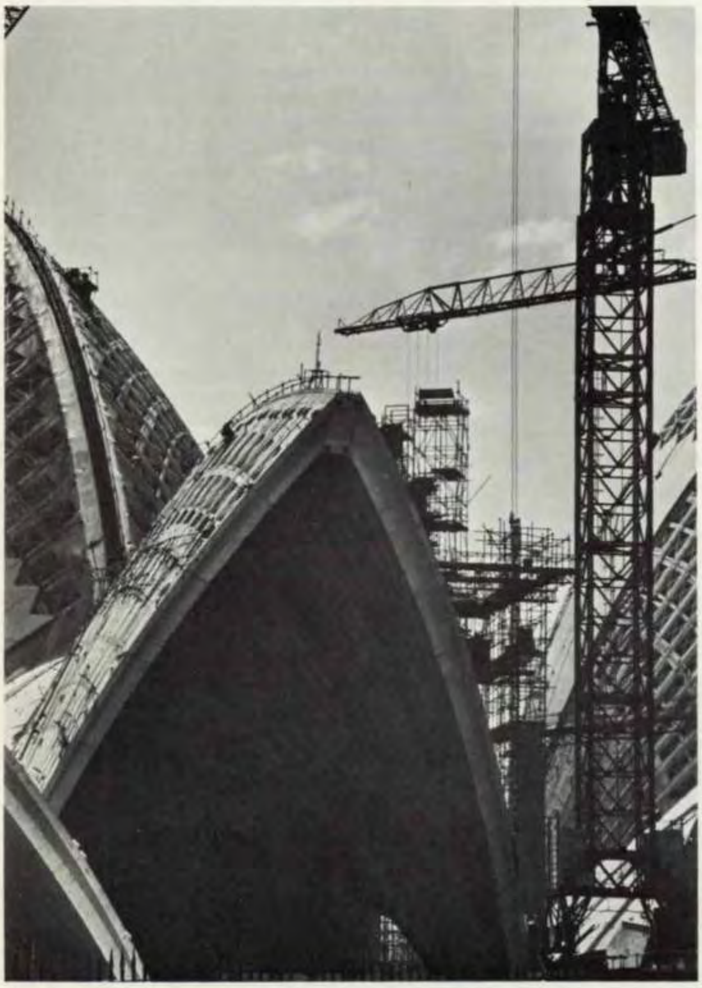
(5) WHITBREAD, R. E. *and* PACKER, Miss M. A. Wind pressure measurements on a model of the proposed new Sydney Opera House. Nat. Phys. Lab. Aero Divn, Report 1049, 1962.

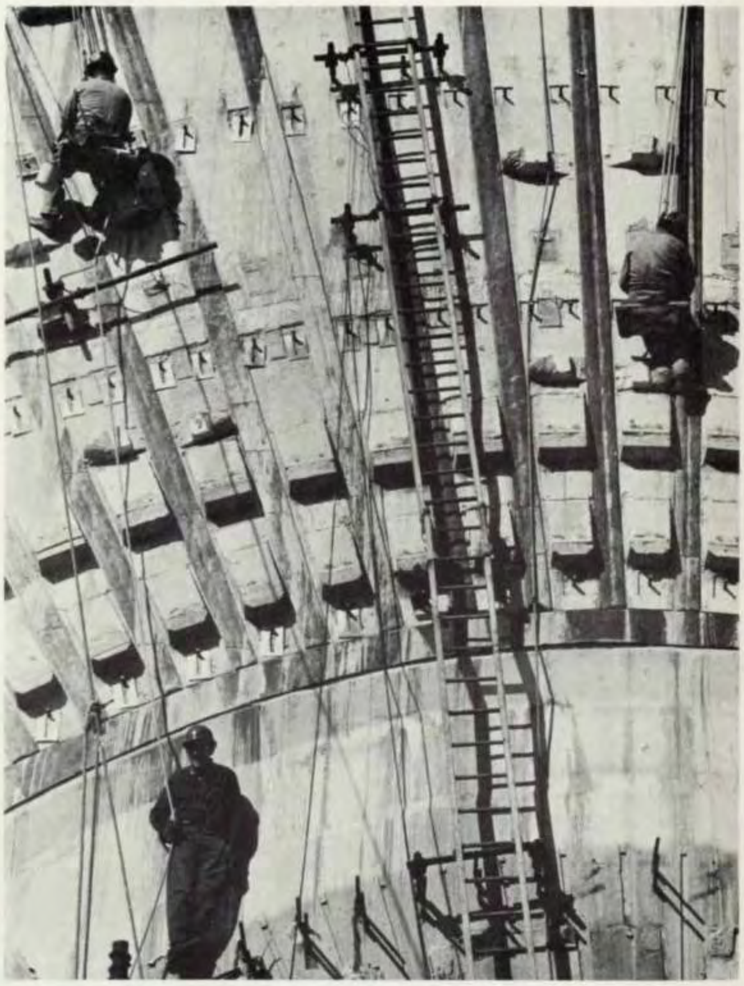
(6) BAKER, A. R. Analysis of two- and three-dimensional steel building frameworks. PhD Thesis, University of Leeds, November 1965.

(7) BRITISH STANDARDS INSTITUTION. BS 153: Girder bridges: Part 3B: Stresses. BSI, 1958.

(8) BASE, G. D. Shear tests on very thin epoxy resin joints between precast concrete units. C & CA Technical Report TRA/366, December 1962.

(9) CARMICHAEL, A. J. The tensile and shear strengths of epoxy-joined concrete specimens. Reports 1 to 4, University of NSW. (unpublished).





 (Photos: Harry Sowden)


The evolution and design of the Concourse at the Sydney Opera House

Ove Arup and
Ronald Jenkins

This paper first appeared in the Proceedings of the Institution of Civil Engineers, *April 1968. It is reproduced here by kind permission of the Council of the Institution of Civil Engineers.*

The structure discussed in this paper, generally known as 'the Concourse' or 'the folded slab', forms a small part of the Sydney Opera House. The architect for the scheme was Jørn Utzon of Hellebæk, Denmark, and the structural engineers were Ove Arup & Partners, Consulting Engineers, London. The structure has a somewhat unusual shape, which was determined more by architectural than by structural considerations. As a rule authors of engineering papers only touch lightly on the development of a design and the aesthetic intention behind it, confining themselves strictly to the structural, constructional and perhaps functional aspects. In the present case, however, the authors felt that this approach would be too narrow, because it would not explain why the structure was given this form. Certainly, functional and structural reasons alone would not have produced it, although they had a considerable influence on it. The first part of the paper will therefore try to explain how the design was produced by the joint effort of architect and engineers.

Introduction

Figs. 3 and 4 (p. 5) show two of the original competition drawings submitted by the architect, on which the location of the Concourse is indicated. In the following an attempt will be made to describe the progression of architectural and structural considerations put forward by the architect and the engineers which led to the chosen design.

In Fig. 4 (p. 5) it will be seen that the architect had originally shown the Concourse supported on a number of columns at midspan. However, when this structure was first discussed between the architect and engineers, the architect asked whether it would not be possible to do without these columns. A typical question, which received the typical answer, that of course it was possible, but would cost a lot of money, and as the columns did not obstruct anything this expenditure might not be justified. The architect then explained that his concept demanded that the architecture should be expressed through the structure, in fact the structure in this case was the architecture; it should be bold, simple, on an impressive scale and of a form which combined sculptural quality with a clear expression of the forces acting on it. This achieved, finishes could be simple: the concrete itself would speak. The area covered by the Concourse was the place where people would arrive by car to the Opera House, and the impact of this vast unsupported roof would be spoilt by centre columns, even if they did not hinder the traffic. He felt justified in achieving the desired architectural effect by spending the money on a bolder structure rather than on expensive finishes.

The solution proposed by the engineers to meet these aspirations was based on the borehole data supplied by the client, according to which firm sandstone would be found 3–4.6 m below ground level, an assumption which much later was proved not to hold good for the crucial southern end of the site. It was also designed to solve the problem of draining this vast area (approximately 7000 m²). The architect wanted the surface of the Concourse roof to be absolutely level, without the customary falls to drain off the water. Instead, the joints between the proposed 1.83 × 1.22 m sandstone paving slabs would be left open to allow the water to seep through. This meant that it would not be necessary, and in fact not desirable, to provide a solid slab at the top; the supporting structure should be formed as a series of channels leading the water towards the two ends of the Concourse but providing support for the sandstone paving slabs along line spaced 1.83 m apart. Fig. 1 indicates the restrictions placed on the cross-section:

(a) Support for paving slabs every 1.83 m.
(b) Channels in between
(c) Total depth of structure should be uniform over the full length of the span, and this depth should be as small as possible.

Fig. 2 shows the longitudinal layout of one portion of the final structure.

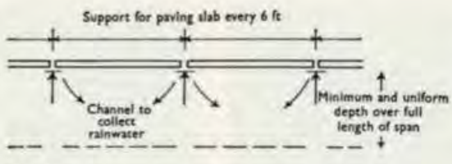


Fig. 1
Basic requirements for cross-section

There are two variants of this layout in other parts of the final structure, with different spans and different depth of structure, and before these dimensions could be fixed the design went through numerous variations, which however did not depart essentially from Fig. 2. It will be seen that if it is assumed that horizontal forces could be absorbed at A by the underlying sandstone, and at C by the substructure, which included a series of reinforced concrete boxes or longitudinal walls, then C–B and B–A could be strutted against each other, creating compression in both struts but reducing the moments. There would be maximum external negative moments at B and C and a maximum external positive moment somewhere in between. A glance at the shape of the 'arch' C–B–E shows that the angle CBA is critical; if it is too large the compression forces will be excessive, and the strains produced by these forces, by the stressing of the cables in these two members, and by creep and temperature stresses, could produce movements which might approach a critical stage. However, after the engineers' proposal had received the blessing of the architect, a preliminary investigation on the basis of the layout as it was then, proved that the proposal was structurally sound.

Confining ourselves now to the main section C–B the task was to design a 'slab' or a series of beams, which would, as economically as possible, meet the requirements in Fig. 1, which could take the negative moments at B and C and the positive at midspan, and which in a dramatic or sculptural way would reflect the variation in the external forces along the span and indicate how they were resisted at each point.

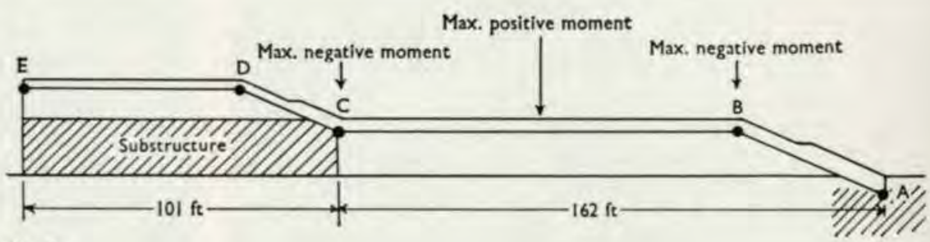


Fig. 2
Basic longitudinal layout

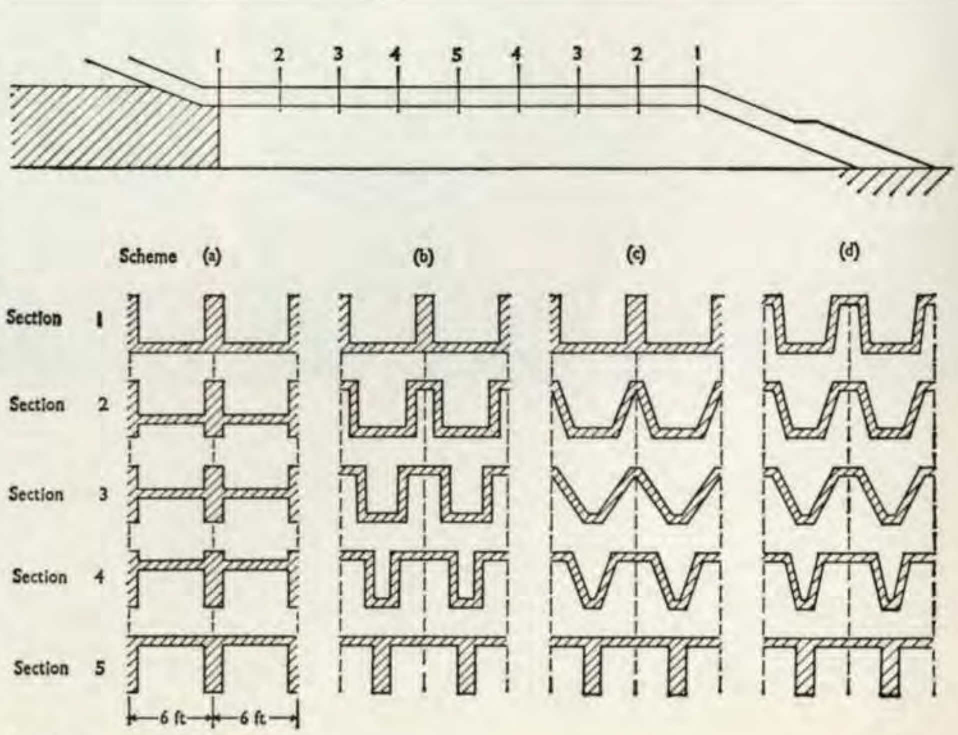


Fig. 3
Cross-sections of various schemes (a)–(d)

This aspiration to have the structure 'truthfully displayed', to achieve 'structural honesty', is of course very familiar to students of architectural theory. It is a declared architectural ideal of long standing, and rightly so. But it must not be taken too literally. Geoffrey Scott showed 50 years ago that this requirement was psychological rather than factual. It has nothing to do with choosing the most efficient structure.

The spectator does not in fact understand the subtleties of a modern concrete structure, whose strength in any case may be hidden from the eye in the form of reinforcement or cables. It is not so much a question of how the structure really acts, but rather of how the spectator thinks it acts, or whether he can relate it to some simple structural facts which lie within his experience. Thus he may be able to appreciate the strength of an arch springing from solid abutments, a cantilever which is strong at its root, a simply-supported 'fish-belly' beam or a fixed beam with haunches producing an arching effect, and this may give him an impression of structural 'rightness'. More subtle effects would be lost on him; they would not form part of his architectural experience.

In this particular case the most economical answer would probably have been a series of box-sections or **I**-beams spaced 1.83 m apart, uniform over the whole length, with prestressing cables catering for the variations in the moment. But this would obviously not have met the architect's request at all. It seemed natural to the engineers, therefore, to seek the solution by exploiting a typical and by now very familiar reinforced concrete form, the T-beam. This can be said to be the best shape to take positive moments in reinforced concrete. And the same shape, only upside-down, is the best shape for negative moments. In this way the desired expression of the variation in the external moments could be obtained by varying the shape from a series of inverted T-beams at the supports to T-beams at midspan – or from section 1 to section 5 in Fig. 3.

Such a solution would make structural sense in reinforced concrete if the formwork could be made reasonably simple, and full use made of the repetitive nature of the job. It would also be appropriate for prestressed concrete if the live load were small compared with the dead load, which was thought to be the case at the time. As it happened, due to the exigencies of the programme, it was not practical to place the paving slabs before prestressing, and these had therefore to be counted in with the live load, making the two about even. This considerably reduced the structural usefulness of the changing concrete sections, and made it impossible to justify the design on economic grounds. But it met the architectural requirements and there was no question of going back to a straightforward box-section.

The question then was how gradually to merge section 1 into section 5 (Fig. 3) in a manner which

(a) Produced a sculpturally interesting soffit
(b) Produced the lightest possible structure for the given depth, i.e. least redundant material
(c) Was easy to construct.

Fig. 3 shows four ways of doing this. In (a) the slab is simply raised through successive sections from 1 to 5. In (b) the walls are moved sideways from 1 through 2, 3, 4 and 5, gradually extending the top slab and contracting the bottom slab. In (c) the walls are gradually twisted, inclining more and more towards each other and reducing the area of the bottom slab until both slabs reach the same minimum, then twisted the other way, thereby increasing the area of the top slab until this covers the whole area, when section 5 is reached.

In these three cases a further variable must be determined before the shape of the soffit is defined, namely the 'speed' of the change in

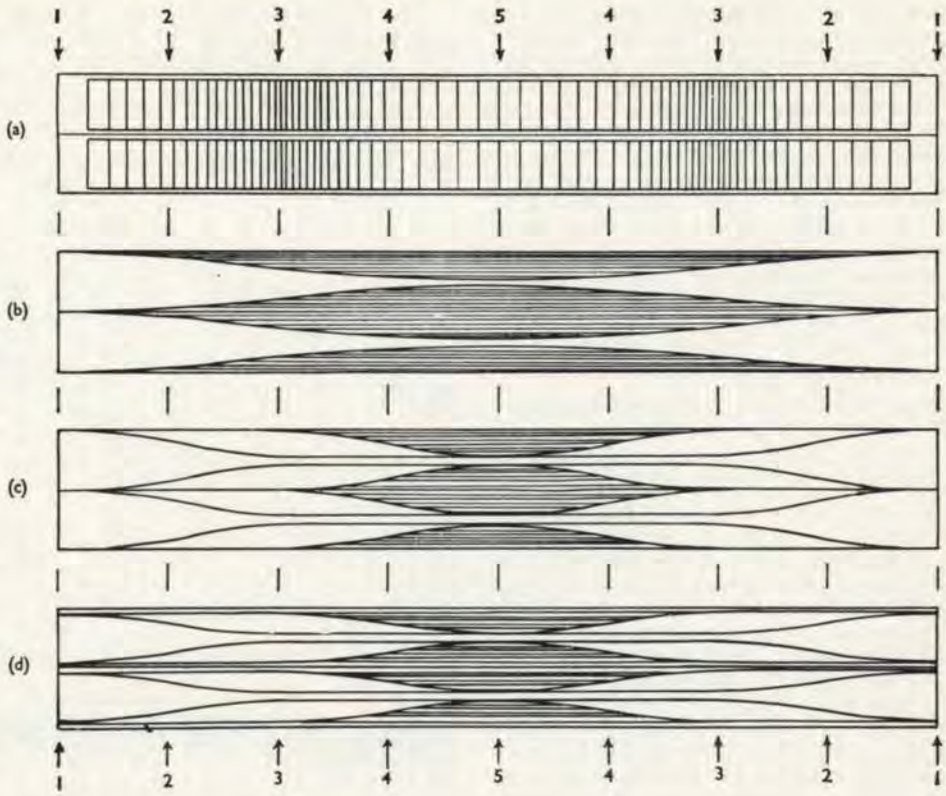


Fig. 4
Soffit of various schemes (a)–(d)

section along the axis of the beam. In order to make the shape as smooth and flowing as possible the engineers decided to make the change in section follow a sinusoidal variation.

In scheme (a) this form of variation would produce a series of beams 1.83 m apart connected by a wavy slab; in (b) there would be 'wavy' beams connected in alternate bays by flat top and bottom slabs, and in (c) the wavy beams would be twisted at the same time. In Fig. 4 (a), (b), (c) and (d) are attempts to show how the soffits would appear. In the engineers' opinion there was a progression from (a) to (c) in aesthetic interest and also in some ways in structural suitability, but unfortunately the complexity of formwork was also increasing. However, as the twisted surfaces of (c) contained straight lines, these could be produced easily enough from straight boards or twisted plywood. This was therefore the solution put forward to the architect, but in a slightly modified form, as indicated in Fig. 3 (d). It seemed to the engineers that scheme (c), seen from below, looked too much like a flat soffit with certain regular hollows scooped out of it. By connecting the hollows together, i.e. by introducing a piece of top slab between the 'beams' of the same width as the beam at midspan, the appearance was more of a series of swelling and undulating beams, and the shape of the beam soffits was repeated in the underside of the top slab. Another minor modification was that the beam sides, instead of being vertical in sections 1 and 5, were slightly slanted to facilitate withdrawal of the formwork.

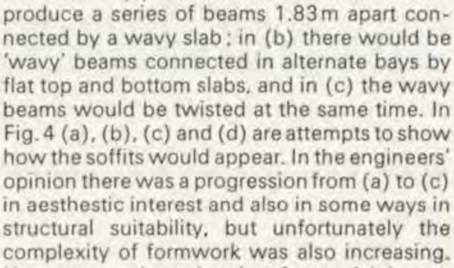


Fig. 5
Proposed round-edged sections

Scheme (d) was at once approved by the architect, and was the one incorporated in the preliminary design submitted to the client by the architect and engineers in April 1958, and approved. The architect had however introduced a further modification in the design, which the engineers were not too happy about. He insisted that the visible corners between the modulating walls or beams and the soffits of top slab and beam should be rounded off, as shown in Fig. 5, and explained that this was very necessary in order to bring out the sculptural quality of the design. The engineers did not dispute this, but were worried about how to produce these rounded corners, and thought it would be very difficult and expensive. It had been their idea that the forms should be made of straight narrow boards forming the twisted surfaces, which would therefore show the familiar boardmarkings characteristic of structural concrete. However, the architect demonstrated on a small model that these board marks and the sharp corners would be out of scale, and that the desired effect could only be achieved by smooth rounded surfaces.

A joint visit paid by the architect and the engineers to the Sydney plywood factory of Messrs. Symonds, who were masters in the manipulation of plywood, confirmed that the architect's ideas would be difficult to realize, and on the return journey from Sydney the designer therefore considered other and more practical ways of effecting the transition from section 1 to section 5 in Fig. 3 (d).

It appeared that there were not so many simple ways of effecting this transition, if one observed the rule that the cross-sections should always be made up of straight lines, which would then produce twisted surfaces which could be made up of plywood. The method proposed in Fig. 3 (c) and (d) seemed to be the simplest possible, i.e. tilting the side B–C (Fig. 12, p.7), rotating it round point B until point C coincided with D, then twisting the side back in the other direction, rotating about D until point B coincided with F. The next simple method (Fig. 6) seemed to be to rotate the side B–C round point B as before, and simultaneously to rotate part of the beam soffit D–C round D, in such a way that the point of intersection C between the two lines moved on a straight line from C to its ultimate destination, point F. The resulting shape of the beam, assuming that the sinusoidal variation of cross-sections was maintained, proved to be very interesting and to possess that roundness or voluptuousness which the architect was looking for, in spite of the fact that there were no rounded corners. Fig. 13 (p.8) shows some typical cross-sections and Fig. 14 (p.8) a dimensioned section of the executed scheme.

After considering this new proposal and making models to judge its effect, the architect wholeheartedly approved of it, adopted it, and had it passed by the Technical Panel.

The engineers, having concentrated their attention on obtaining an architecturally-interesting solution which could be produced with fairly simple formwork, had at that stage possibly given too little weight to one possible disadvantage of the last scheme compared with that in Fig. 3, namely that the 'kink' in the side walls might increase the internal bursting stresses produced by the bending stresses resulting in an increase of ordinary reinforcement, thereby adding to the difficulties of compacting the concrete. But this was only a minor snag compared with many others which emerged during the detailed design and the execution of this work. For one thing, the assumptions on which the design was based underwent various changes, all for the worse. It was found, for instance, that the underlying sandstone dipped down at the southern end of the site, making it doubtful whether the safe bearing which the design called for could be provided at this end. Then the architect changed, at the Technical Panel's request, the ratio between treads and risers of the steps, flattening the slope of A–B (Fig. 2). As pointed out earlier, the angle C–B–A had a vital influence on the horizontal forces which had to be absorbed. For these reasons it was found desirable to introduce tie-beams between the foundations at A and C. This added to the cost, but put the design on a much sounder basis, and part of the cost was offset by the fact that the prestress produced by the cables in the tie-beams made it possible to reduce the number of cables in the folded slab itself.

All these changes naturally delayed the completion of the detailed drawings which were urgently needed on site, and further aggravated the almost impossible situation which was created by the client's insistence that work should begin on site early in 1959, long before the brief – let alone any finished and dimensioned drawings – had been completed. The situation was not improved by the contractor's insistence that his programme demanded an early start on exactly this particular part of the job. Add to this the difficult nature of the job, complicated or unusual formwork, narrow sections packed with steel, etc., and the contractor's unfamiliarity with prestressed concrete, and it is no wonder that the atmosphere on the job deteriorated and the workmanship suffered.

A description of the snags which developed and of how they were overcome would perhaps be useful but falls outside the scope of this paper. But it may be of interest to mention another complication which was happily avoided, because it concerns the design, and it throws some light on the somewhat different points of view of architect and engineers.

It arose from the fact that a part of the Concourse slab (the part under the restaurant) was raised a few steps over the rest. It was part of the architect's philosophy – to use a now popular phrase – that the structure, i.e. the

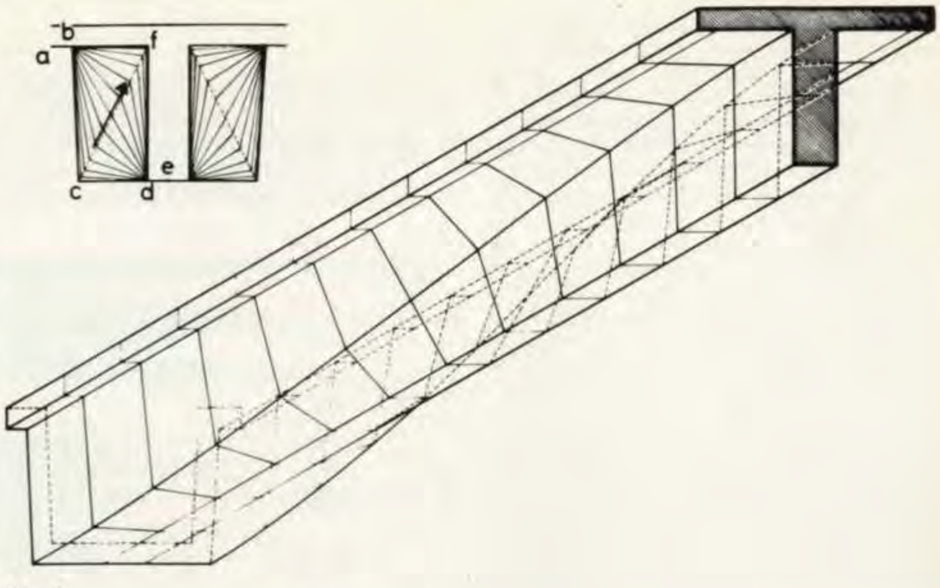


Fig. 6
Isometric view of executed scheme

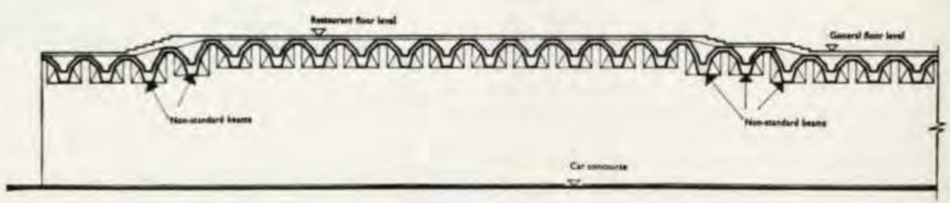


Fig. 7
Cross-section through proposed slab

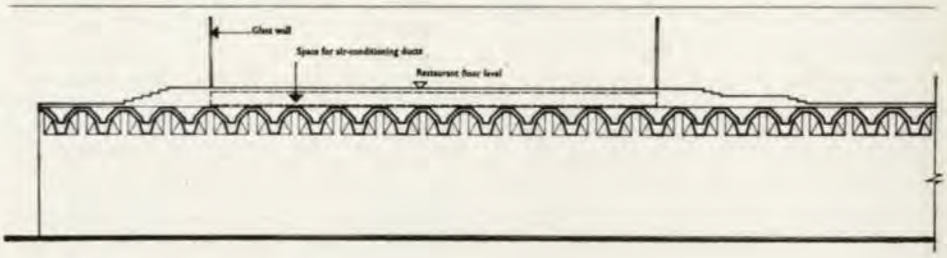


Fig. 8
Cross-section through executed slab

shape of the slab as seen from below, should register this fact: one should be aware of what happened above, just as one should be aware of the forces acting on the slab. The cross-section in Fig. 7 shows what the architect wanted to do: the beams under the higher portion are lifted up and there is a gradual transition to the normal level, more or less following the steps above. It was difficult to argue that this could not be done, although it posed tremendous problems, because the five special beams were unsymmetrical in cross-section and the prestressing would create torsional movements which would have to be absorbed by the adjoining, already fully-stressed, beams. This would require structural additions and might even prove to be almost impossible – apart from the fact that it would upset the whole arrangement of stressing two adjoining beams at a time, and would require five sets of special and more complicated forms, thereby invalidating the excuse of repetitive formwork.

The engineers' view was that even if the architect was correct in preferring his solution from an aesthetic point of view – which they did not dispute – the very considerable cost, and the disturbance it would cause in an already critical situation, would be too high a price to pay for something which after all would not be missed by anybody. However, the architect was insistent and the engineers were bracing themselves to attempt a solution to the problem when the heating engineers intervened with a demand for space over the slab in which they could accommodate their pipes and other services. This clinched the matter: by keeping the beams at the same level as in Fig. 8, the desired space would automatically be created and everybody was satisfied!

Figs. 9 and 10 show the appearance of the Concourse slab from underneath and from the side, and Fig. 11 is an aerial view of the finished slab without the pavement.

This account of the development of the design is necessarily brief and deals only with the typical case. Amongst other things it leaves out the considerable difficulties in creating the correct boundary conditions, especially for the cross-beams spanning the openings in the supporting wall at the north end, but these are dealt with in the next part of the paper.

Structural design

The Concourse is a folded-plate structure in prestressed concrete and the analysis was quite conventional. Some unusual features were present, however, because it was not simply a bridge but had also to have the architectural attributes already described.

The depth/span ratios were low and the shape was not ideally suited to prestressing. The structural design was, thus, mainly concerned with keeping within the working stresses under all conditions. The cross-sections continuously varied according to geometrical rules. For this main reason most of the numerical work was programmed for the electronic digital computer. The analysis was done and the working drawings prepared in 1959.

The remarkable feature was the extremely flat angle of 19½° of the leg from A to B (Fig. 11, p. 7). This made the portal part of the structure (A, B, C), very sensitive to the phenomena of concrete movements. Time, temperature and load gave many combinations to investigate and reinforced the case for computer working.

A general description of the whole structure will now be given.

The part denoted by RC substructure in Figs. 10 and 11 (p.7) consisted of two-storey box-like structures which received the prestressed tie-beams in a floor at that level. The compression in the tied portal due to the thrust at the foot, A, was taken up at the other end by shear walls in the substructure.

The couple thus imposed on the substructure required special measures for stability, but was a greatly reduced problem compared with the original idea of thrusting against the rock, because the indicated rock level was well below the tie level.

The substructure was not there just for the structural purpose described. The shear-wall spacings were determined by the various uses of the rooms. A certain amount of structural irregularity was a small thing compared with the large spaces required for main stairway entrances and especially for the service road, 12.8 m wide, which entered the Opera House at tie-beam level at the centre of the Concourse. These boundary conditions at the north side of the portal are shown diagrammatically in Fig. 10 (p.7).

The upper reaches of the Concourse shown typically from C, D to E in Fig. 11 (p.7), were subject to several varieties of spans, slopes and flats. The figure indicates that where visible to the public, the architect required the 1.83 m wide, varying section, folded slabs to flow continuously into the upper reaches. Because of the reduced depth the most critical point was found in the upper sloping part.

For an ordinary continuous beam one would have introduced something like a concrete hinge over the intermediate support at C. However, the vertical reaction at C was combined with a horizontal reaction of the order of 203 tonnes/1.83 m wide Concourse beam. A hinge under these conditions would have required costly mechanical devices. It was decided to make the beams monolithic with the shear walls, which in the region of C had thus to receive the vertical reaction, the portal compression and a couple equal to the difference between the end moment of the portal part and that of the upper part of the Concourse.

Transom beams, in one case of considerable size, were introduced across the wide openings mentioned above.

The transoms were in a state of vertical and horizontal bending and torsion. The torsion arose from the beam end moment differences which depended on the combinations of temperature and loading. The minimum would be obtained if maximum clockwise and anticlockwise moment differences were numerically equal. By preloading and other devices the upper spans were made as far as possible to bring about this optimum condition.

The stresses in the transoms resulted in deflections and rotations, but these were of a small enough order not to influence the assumed fixed end condition of the Concourse. One meets a parallel case in the edge beam of a cylindrical shell. When the beam is of normal size it makes no material difference if one takes into account its torsional rigidity or assumes it does not twist.

The portal part of the Concourse contained 47 folded slab units, 1.83 m wide. The slabs were 178 mm thick: there were 21 units 50 m in span and 1.37 m deep; the remainder were 41.5 m in span and 1.14 m deep. Both types were designed in a similar way. The longer span has been selected for description in the paper.

With the sloping leg at such a flat angle, a structure with unusual sensitivity to concrete movements seemed to provide a good opportunity for correlating calculated and measured strains and deflections. The distance from London, where the design was done, made the site measurements less extensive and accurate than was desirable for the exercise. The authors do not believe that the results add anything to present knowledge (a common finding unless the instrumentation is good) and the correlation will not be given in this paper.

Long-term movements were important. The construction of the portals was begun in November 1960, and finished in January 1963. The laying of paving slabs, which are a permanent superimposed load, may not commence until 1968/9. The application of full live load will be a rare and short-term event.

Ultimately the concrete of the portals will be shielded from direct sunlight by the paving. The average temperature condition will be when the temperature of the portal concrete is the same as that of the tie concrete. Long-term factors are that the paving should be quite flat at average temperature when there are few people on it and the possibility of further creep due to changes of stress from the weight of the paving.

The control devices used to make these structures largely independent of concrete movements not accurately known will be described. However, for the sake of the correlation the designers made some research into published experimental information and made use of what they thought was the latest at the time, not very different from that given in the British Standard Code of Practice (CP115:1959).

A pair of connected folded slab units 3.66 m wide, were cast at a time. A gap, to be filled in later, was left between one pair and the next pair. In this way stressing operations could be carried out on a pair of connected units without disturbing the adjacent units.

The pair of ties corresponding to a pair of folded slab beams passed on opposite sides of the T-sections at A and were connected by a cross-head. Hydraulic ship jacks of 203 tonnes pressure were introduced into the gap between the cross-head and the foot of the portal. Two ship jacks were used and one other was kept as a standby in case of breakdown. Also in the gap was a pair of steel wedge assemblies. The details of the jack and wedge apparatus are shown in Figs. 11 (p.7) and Fig. 15 (p.8). The former also shows the profiles of the 28.6 mm diameter strand post-tensioning cables.

Load cells or dynamometers were used with the hydraulic jacks and the prestressing jacks so as to obtain more accurate force measurements than could be relied upon from pressure gauges. The order of prestressing and jacking was carefully worked out so that the structure could be converted from an unstressed, inert state supported by soffit props to a fully-stressed free-standing condition without at any stage exceeding the permissible design stresses at transfer. Creep and shrinkage losses in the portals were made good by periodic

Fig. 9
Completed structure 1

Fig. 10
Completed structure 2

Fig. 11
Aerial photograph of site showing Concourse area

rejacking, which meant moving the ship jacks around quite a lot.

The folded slabs were cast to true final shape. The knee at B could be maintained to correct height by the ship jacking and geometrical non-linearity could thus be avoided. The jacking was not simply to allow for concrete movements. The calculated force obtained optimum stress conditions in the portals. The jacks were, in fact, a second method of prestressing.

The tie-beam had been previously prestressed to a concrete compression of about 9 N/mm². Thus the operation of the jacks decompressed the concrete. Data of creep recovery for the intended correlation were found to be too scanty for consideration.

A case could be made for the use of Freyssinet flat jacks throughout in place of the ship jacks and wedges. The ship jacks were ordered when it was thought the jacking might be against rock instead of ties. The jack travel might then have been quite large to an extent that could not be predetermined.

To deal with the possibility of extra creep due to the paving load the intention is to stack the paving slabs in transverse lines across the Concourse. The jacks will then be put into position again and used to compensate for any loss of thrust and maintain the correct level at B. The jacking gap will be concreted in when everything has settled down and the gap left between every pair of units will be made good. The paving will then be laid true to level at a period of average temperature.

It should be mentioned that since there was an odd number of folded slab units of the long span type, one unit had to be made by itself with a gap on each side.

Fig. 12
Construction sequence

Materials

Concrete
The fine aggregate was a uniformly-graded Cronulla sand with approximately 4 per cent moisture content. The coarse aggregate was Prospect Blue Metal, a crusher run aggregate, uniformly graded with a maximum size of 22 mm. A concrete mix was designed whose proportion by weight was 1:1.19:2.43. The water/cement ratio was 0.39. An additive, *Darex WRDA*, was included at the rate of 2.2 kg/m³ of concrete mixed.

The authors' specification called for a minimum cube strength of 48.3N/mm² at 28 days, and prestressing was allowed to begin when field cubes reached an average strength of 41.4 N/mm².

The permissible working concrete stresses adopted in design, were:

	N/mm²
Bending compression	15.5
Bending tension	1.5
Shear stress	1.2
Local bond stress	1.2
Average bond stress	0.8
Principal tensile stress	1.1

The prestressing strand was augmented with mild steel reinforcement where tensile stresses were found.

The coefficients assumed for calculating deflections were:

Young's modulus at transfer	34.5 kN/mm²
Long-term Young's modulus	17.2 kN/mm²
Young's modulus in tie	34.5 kN/mm²
Shrinkage strain	0.03%

Prestressing
The prestressing equipment for 28.6 mm diameter strand was by Gifford-Udal and a dynamometer of Swedish construction was added for accurate measurement of stress. The 28.6 mm diameter prestressing strand, which had then been only recently marketed, had a guaranteed minimum tensile strength of 82 tonnes. It was housed in *Kopex* ducting with an inside diameter of 38 mm.

The mechanical properties of the strand, assumed in design were:

Young's modulus, tangent	165 kN/mm²
Secant modulus at 63 tonnes	131 kN/mm²
Friction constant between cable and ducting	$\mu=0.30$
Wobble constant	$K=0.0010$

The use of the calibrated dynamometers at the live end and dead end of several cables showed $K=0.0013$ and $\mu=0.25$. It was found that the change of these constants from those assumed did not materially affect the analysis.

Loss of prestress
The relaxation of the strand cables was determined by experiments carried out at the University of Sydney.

Loss of prestress partly depended on the distance from anchorages, the angle turned through and the concrete stress at the centroid of the cables, under dead load conditions without paving. The loss at transfer was distinguished from the ultimate loss after several years under dead load.

The ultimate loss averaged about 33 per cent. The approximate average contributions were 19 per cent due to friction, 5 per cent due to creep and shrinkage, 5 per cent due to anchorage slip and elastic shortening and 4 per cent due to relaxation of the strand. The latter was reduced from 6 per cent by holding the maximum force of 63 tonnes for five minutes before wedging-off. The high friction loss was partly due to the large proportion of cables stressed from one end only through restrictions in fitting stressing jacks into position at both ends. The restrictions arose from having all the anchorage in the upper hollowed-out part of the beams, for the visible undersides could not be marred by the making good of anchorage pockets. The elastic shortening mainly arose from the thrust of the ship jacks. The prestressing cables were grouted before the ship jacks were brought into full operation.

Thermometers
The temperature difference between the folded

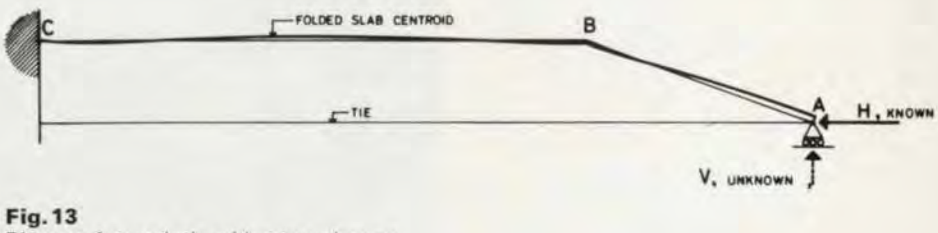


Fig. 13
Diagram for analysis with one unknown

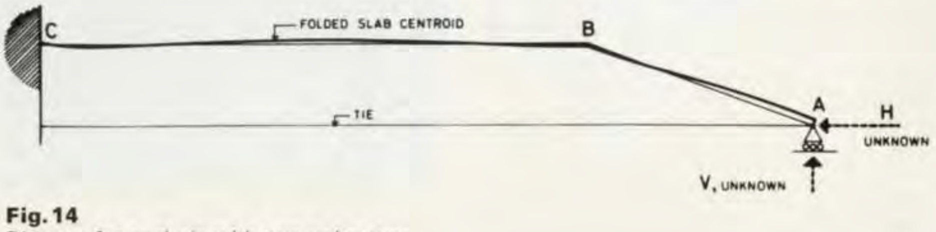


Fig. 14
Diagram for analysis with two unknowns

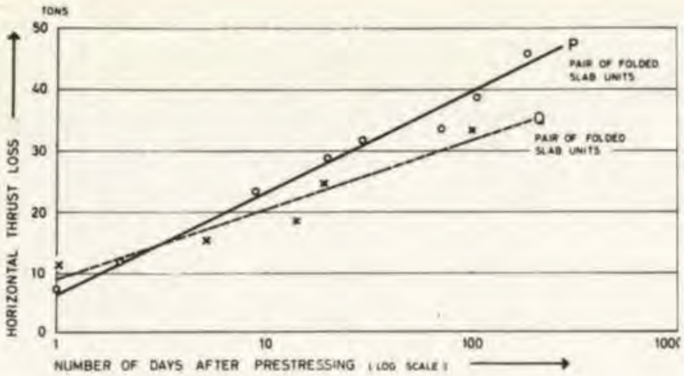


Fig. 15
Relaxation/time curves

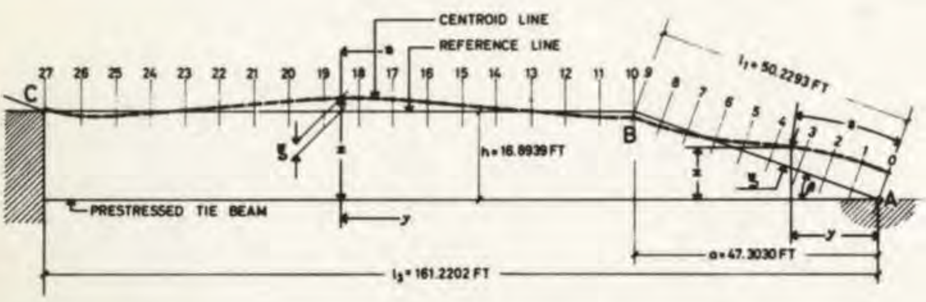


Fig. 16
Diagram for structural analyses

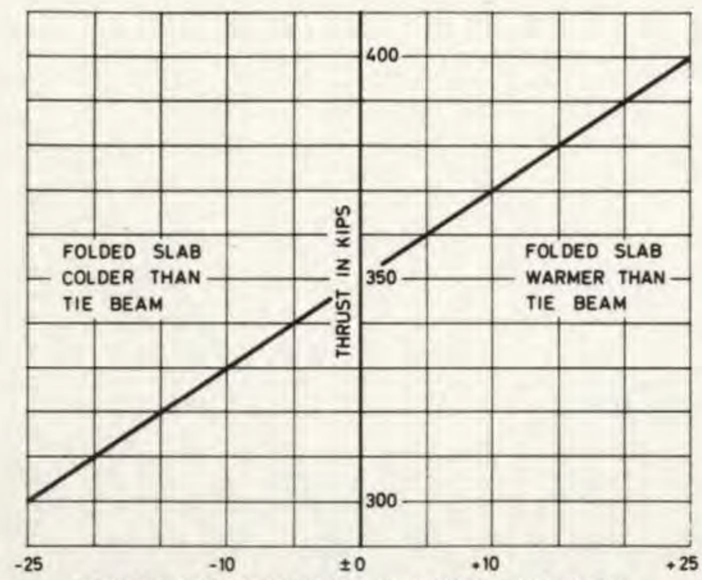


Fig. 17
Gain or loss of thrust due to temperature

slab and the tie had an influence, as will be explained, on the forces applied by the ship jacks. Eight thermometers were located in each pair of tie beams. The thermometers were immersed in water in specially constructed temperature pockets and read just before the jacking operation. The temperature range used in the structural design was ±14°C.

Portal design

The portal foot at A rested on a foundation through lubricated sliding plates.

Two cases of statically indeterminate structures were analysed by the influence coefficient method. The analyses were programmed for the electronic digital computer to eight significant figures. In view of this the simplest released system was adopted, that of cantilevering the whole folded slab structure from C. Bending and direct strain were taken into account, but not shear strain. However, the program was arranged to give shear forces. Part of the data was the prestressing tensions and their slope, which were taken into account for resultant shear on the concrete.

The first case is shown diagrammatically in Fig. 13. At A, the horizontal thrust is known and the vertical reaction is the unknown. The second case is shown in Fig. 14. The unknowns are the vertical and horizontal reactions at A. The designers had to consider the effects of concrete movements on these indeterminate structures. In the first case, the horizontal thrust was to be brought up from time to time to a known force over a period of several years.

There was no doubt that shrinkage, creep, elastic tie extension, and the loss of the horizontal component of folded slab compression would be taken up by jack travel. The vertical component of the compression in the sloping leg, naturally, came into the analysis. When it came to rejacking, it was found, as expected, that when the thrust was just taken up there had been a loss. Fig. 15 shows two plots for loss of initial thrust against days on a logarithmic scale. The differences in the curves were due to differences in time from casting the concrete to stressing operations.

It is well known that if the modulus of elasticity *E* is constant throughout, which would be the case if creep is proportional to stress, it does not enter into load analysis because its value comes in the denominator on both sides of the equation of relative movements due to loads and those due to the unknowns (redundants). In fact, influence coefficient methods amount to finding the redundants by this equation. If there is more than one redundant, the equations are simultaneous, as in the second case.

When there was a relative temperature change while the horizontal thrust was held on the wedges, the second case of two unknowns arose. Further, in this case there were relative movements in the released system which comprised the change of vertical height of the sloping leg and the relative changes of horizontal length of folded slab to the tie-beam, which had to be eliminated to restore continuity by redundant actions. It is therefore evident that the value of *E* comes into only one side of this equation and can not be cancelled. The folded slabs were exposed to the air and the tie beams were not. The daily alterations of temperature were small, but there were sometimes larger temperature differences of the same sign over periods of several days to a week or more.

The temperature differentials were regarded as short-term events, and the *E* value used was 34.5 kN/mm². Thus, in the judgement of the designers, creep due to temperature stresses was ignored.

As regards the application of the permanent paving load, the structure will still be under the wedge and jack apparatus, and any further creep will still be taken up. If it is presumed that creep will take place where the compressive stress is increased but not where it is decreased, the creep will be resisted by peculiar stress distributions over the cross-sections, unless the creep recovery factor is equal to that of creep strain which seems unlikely. These are stress history matters on which published research was entirely lacking.

The loadings taken for design were:

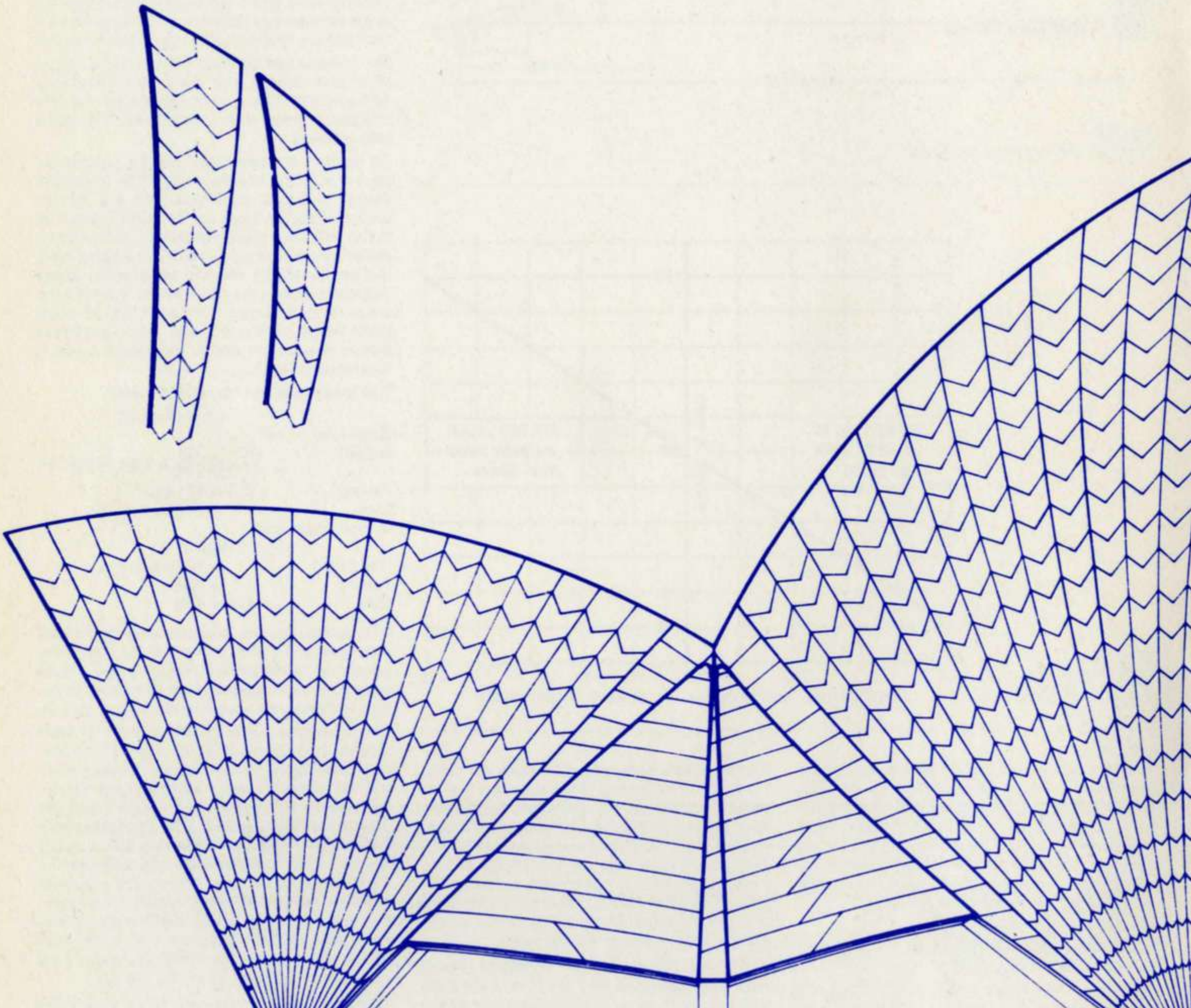
	kg/m per unit
Dead load or self weight	$= \frac{231}{215} \times$ area of section
Paving	= 171 kg
Live	= 272 kg
In plan these are:	
	kg/m²
Dead load	= 1025 average
Paving	= 308
Live	= 488

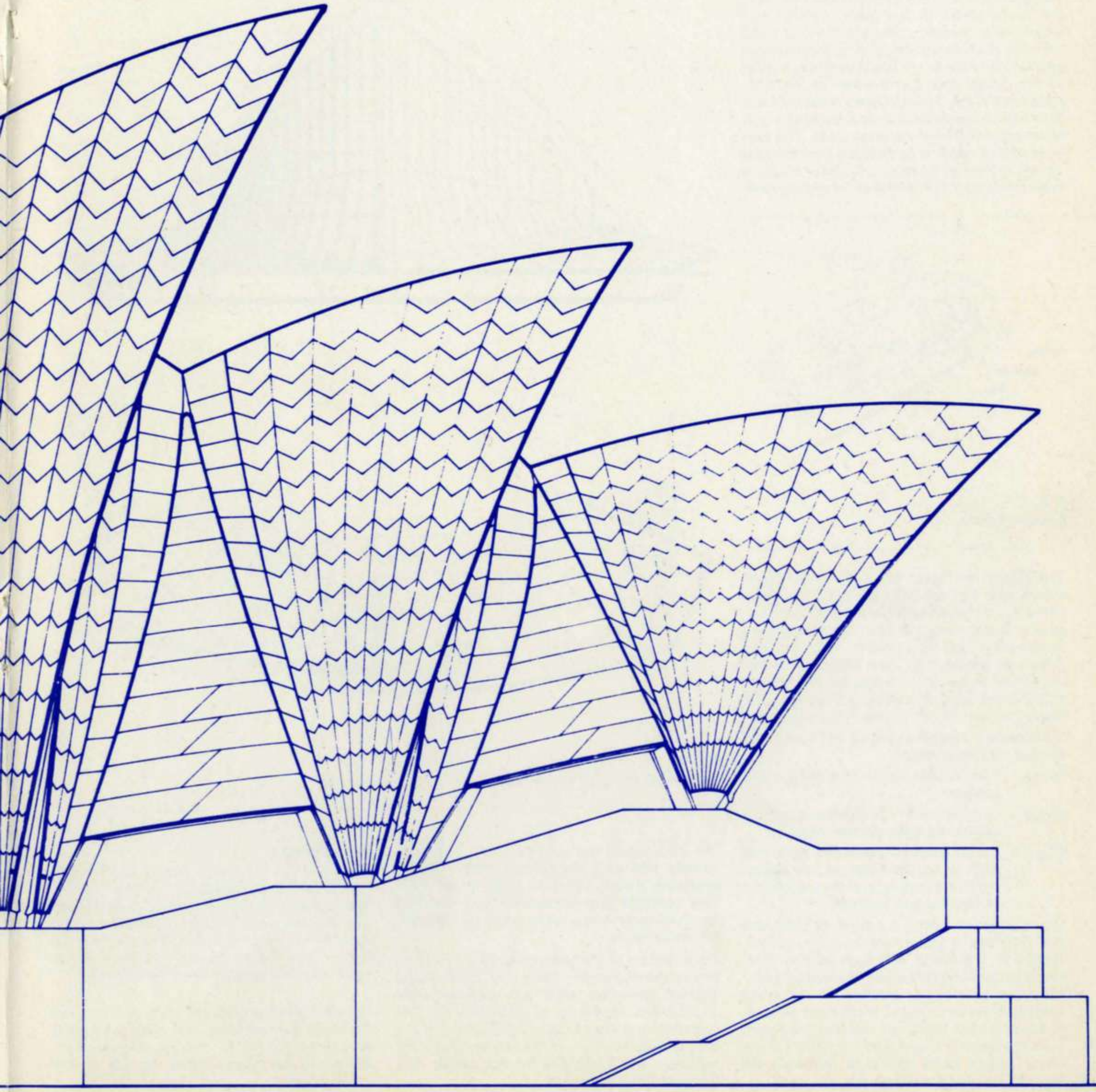
Integrations for the analysis were performed numerically in the computer by summing according to repeated Simpson's rule. These were both total integrals over the whole structure and integrals from A to the interval points.

Cross-sections were drawn to scale at each interval (numbered 0–27 on Fig. 16), to provide dimensions for the two purposes of making the formwork and a computer run to give the area *A*, the moment of inertia *I* and the distance of the centroid, ζ in the figure, from the straight reference lines, AB, BC, at each cross-section. Due to certain changes after the cross-sections had been drawn, some unequal intervals appeared, which required modification to the Simpson coefficients in their regions. From this preliminary program, the co-ordinates of the true axial line (centroid) at each interval were given, denoted by *y*, *z* in Fig. 16.

The graph used on the site to determine the appropriate jacking force at various temperature differentials is shown in Fig. 17.

Computer runs, for the two spans, were made for 19 conditions. These comprised transfer, self-weight and paving, and live load; maximum and minimum temperature differences; and jack thrusts ranging from 203 to 376 tonnes. The latter were for determining how much loss of thrust could be tolerated and to what extent over-jacking could be employed to compensate for future losses.





The Sydney Opera House Glass Walls

David Croft and John Hooper

This article is a combination of a paper given by David Croft at the Fourth Australasian Conference on the Mechanics of Structures and Materials, Brisbane, August 1973 and the paper by David Croft and John Hooper which appeared in the Structural Engineer, *September 1973. The latter material is reproduced by kind permission of the Council of the Institution of Structural Engineers.*

Fig. 1
View from harbour showing glass walls A4 (right) and B4 (left)
(Photo: Harry Sowden)

Introduction

The Glass Walls of the Sydney Opera House is the name given to the glass surfaces that enclose the openings between the roof shells and the podium structure. A comprehensive account (see pp. 4-19) has already been given of the design and construction of the main structures of the Sydney Opera House. At that time one of the major technical problems outstanding was that of the glass walls. The purpose of this paper is to describe how the final design evolved and, in particular, how solutions were found to the problems of the glazing itself.

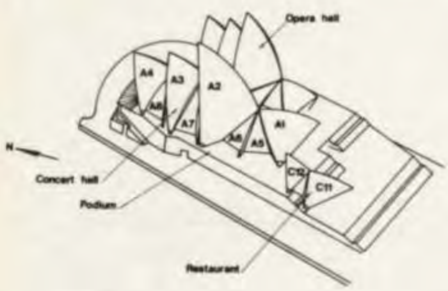


Fig. 2
General layout

The idea of an Opera House for New South Wales was first officially promoted in 1954, and the site chosen was Bennelong Point projecting into Sydney Harbour. An international competition was held and in 1957 the first prize was awarded to Jørn Utzon, who was appointed to be the architect for the project. In the same year, Ove Arup & Partners were appointed as consulting structural engineers.

Construction started on site in 1959 and was divided into three stages:

- Stage 1 Construction of the foundations and podium
- Stage 2 Completion of the podium structure and construction of the shells
- Stage 3 Construction of the louvre walls and glass walls, the auditoria, the cladding to the podium and the installation of services and finishes.

Utzon resigned from the project in 1966 and was replaced by the architectural firm of Hall, Todd and Littlemore. Although by this time, the construction of the shells was almost complete, no satisfactory solution to the glass walls had yet been found. Numerous alternative geometrical forms and materials had been investigated during and after the design of the shells; these earlier schemes, however, are beyond the scope of this paper.

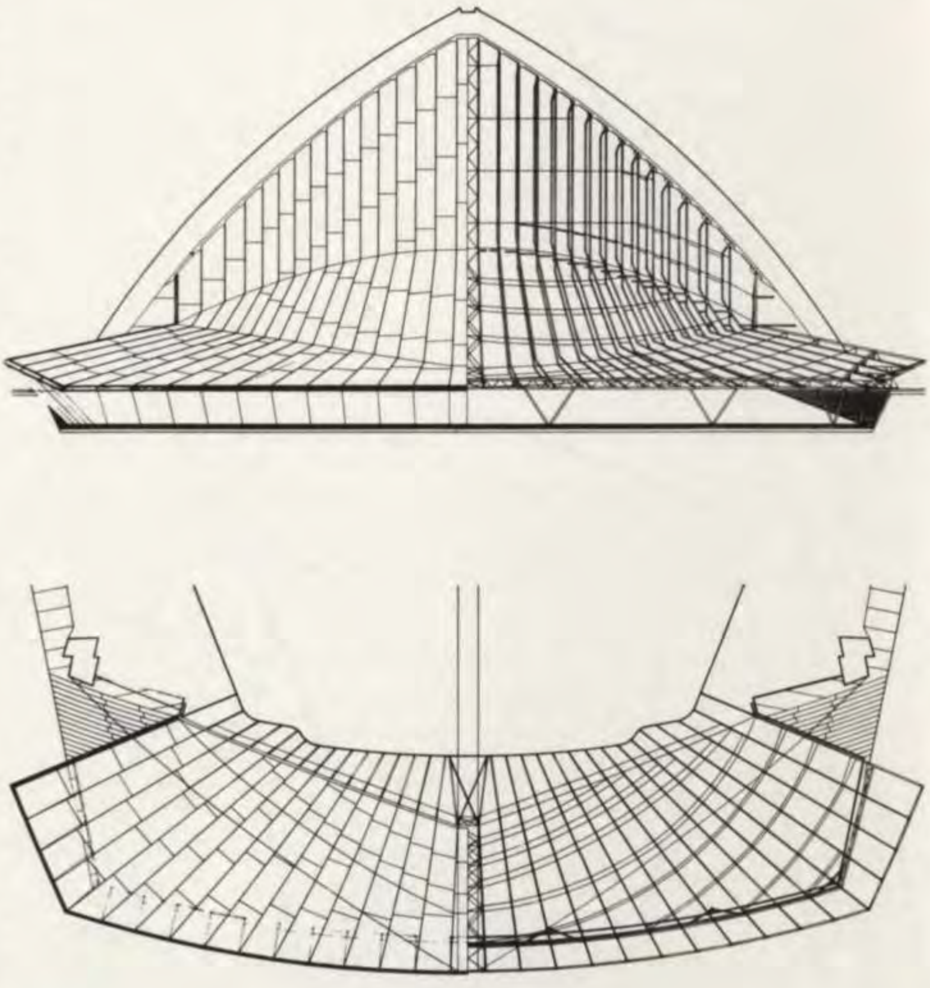


Fig. 3
Glass wall A4: plan and elevation

0 5 10 20m

The concept of the scheme finally selected, namely that of a continuous glass surface enclosing a steel structure, dates from 1967. This concept was developed and involved much research into a wide range of materials and techniques.

Construction of the glass walls began in 1970 and was completed in 1972 (Fig. 1). The total cost of the glass walls was approximately £1,900,000, made up of £300,000 for the concrete and steelwork, £1,100,000 for the supply and erection of the glass (including sealing), and £500,000 for the supply and fixing of the bronzework.

Terminology

The layout of the Opera House is shown in Fig. 2. The main buildings are the Concert Hall, Opera Hall and Restaurant which stand on the podium substructure. The Opera Hall is geometrically similar to the Concert Hall, but smaller, and the shells are numbered in the same way but prefixed with the letter B.

The glass walls are referred to by the names of the shells they enclose. Each shell has a glass wall except A2, A3, B2 and B3 which are connected to the shells below by the bronze louvre walls.

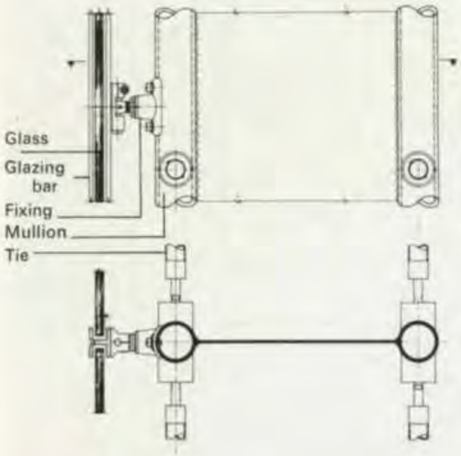


Fig. 4
Detail of glazing and structure

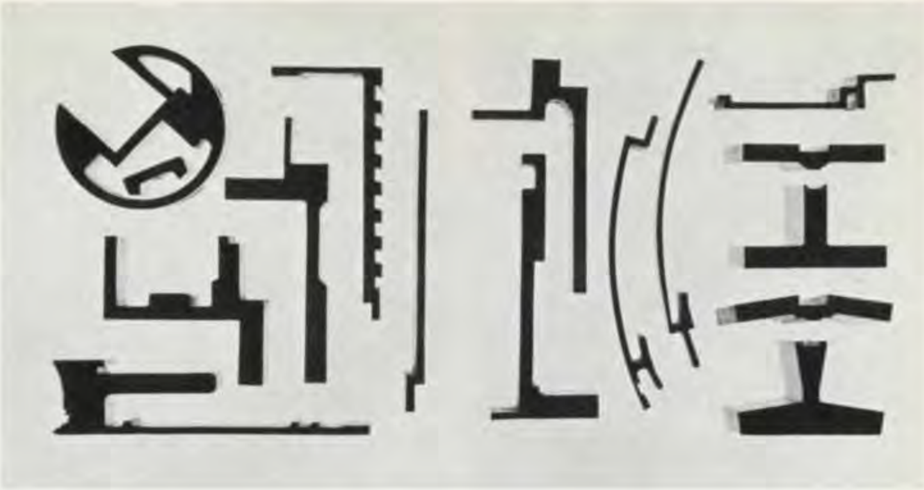


Fig. 6
Bronze extrusions (Photo: Harry Sowden)

Fig. 5
Detail at junction of cylinder and cone (A4) (Photo: Harry Sowden)

The shells themselves are not strictly shells in the structural sense. On the Concert Hall the main shells A1, A2, A3 and A4 are made up of ribs which spring from the pedestals on the podium, east and west sides meeting at the ridge beam at the top. The side shells A5, A6, A7 and A8 span between the main shells on each side. Except for small transitional warped surfaces the outer surfaces of all the shells are segments of a single sphere. The Opera Hall complex is similar and the Restaurant has two main shells C11 and C12 and a side shell C13.

Glass Walls A4 and B4

The design of the glass walls was carried out in parallel with their construction. This was necessary, as time was running out, and weather-proofing the building was critical on the overall construction programme. Design work was, therefore, initially concentrated on A4 (Fig. 3), bearing in mind that the details as they evolved would also have to apply to the other walls.

Choice of glass

The main requirement was for a safety glass that could be cut to shape on site. Toughened glass was rejected in view of the variety of shapes and sizes that were required and the fact that the toughening process would have had to have been carried out after the sheets were cut to exact shape.

Laminated glass was therefore chosen, although at that time there was little information available on its use in buildings. The necessary research programme into the technology of the glass is described later.

The laminate finally selected consists of a layer of clear plate or float glass and a 6 mm layer of tinted glass, together with a 0.76 mm thick interlayer of clear polyvinyl butyral. In order to achieve the precise colour required by the architect, the tinted glass (referred to as 'demi-topaze' by the manufacturers) was produced by the very traditional process of pot-casting. This process is described in greater detail elsewhere.[2]

Two thicknesses of clear glass were used, giving a standard laminate thickness of 18.8 mm and a thicker section of 20.8 mm which was used in certain areas where greater strength and stiffness were required. The maximum sheet size was approximately 4.0 m by 2.1 m.

Glass support system

In the main surfaces, each glass sheet is supported along its two 'vertical' sides by glazing bars, and the top and bottom joints are filled with silicone rubber sealant. The glazing bars were extruded from manganese bronze and in their 'standard' form consist of a T-section and a cover piece screwed on after the final positioning of the glass. The combined sections act together as an I-section (Fig. 4). The glazing bars follow the lines of the supporting structure inside and each glass sheet is held vertically by two steel pins projecting from the flange of the T piece. Fig. 5 illustrates the details at the junction of the vertical and inclined glass surfaces on A4.

For the other surfaces, e.g. the view windows, the glass support system had to be modified. In all, over 40 different shaped bronze extrusions were used; a few of them are shown in Fig. 6.

Adjustable fixing bracket

The glazing bars are attached to the structure by means of fixing brackets at roughly 0.9 m centres. These had to be adjustable to accommodate the geometrical variations in angle and distance between structure and glass. They also had to take up the tolerances in the fabrication and erection of the structure. Allowing for the difficult fabrication involved and after discussions with the main contractor, it was accepted that even with the sophisticated surveying techniques they had developed, any point on the erected structure could be as much as 15 mm out of its theoretical position. On the other hand, the distances between adjacent glazing bars had to be correct to 1.5 mm so that the glass sheets would fit.

The fixing proved to be quite a complex piece of machinery (Fig. 7) and advice was sought from the aircraft industry. The design was developed in conjunction with Hawker de

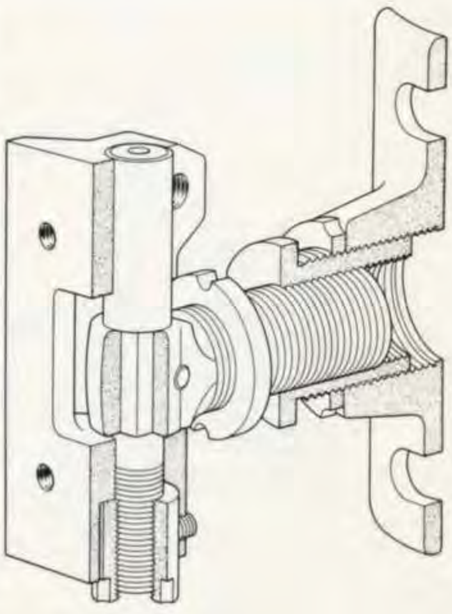


Fig. 7
Detail of adjustable fixing bracket

Havilland (Aust.) Pty Ltd and the fixings, of which there were 2310, were manufactured by them. The material used was aluminium bronze which offered strength together with resistance to stress-corrosion and fatigue, and was also suitable for casting and machining.

The structure

Steel was chosen for the main structural material on account of its strength and stiffness. The standard elements, or mullions as they came to be called, were fabricated from two parallel 90 mm diameter tubes at 530 mm centres joined by a 6 mm plate web. This section had the advantage that the geometry could be solved along the centre line of the outer cord and standard connection details could be developed that would apply to the whole range of orientations that would occur.

Connection to shell ribs

One of the critical details of each of the walls was the method of connecting the mullions to the shells. The position of the cables in the prestressed ribs prohibited any form of drilling into the rib to make a fixing. However, during the design of the shells, certain ribs had been chosen to support the glass walls. These had been specially strengthened and extra holes cast into them to allow for subsequent fixing of the glass walls.

Naturally, these holes did not coincide with the positions of the mullions, and it was therefore decided to cast on to the rib a strip of in situ

Fig. 8
Detail of corbels (A4)
(Photo: Harry Sowden)

Fig. 10
Glass wall A4: structure (Photo: Harry Sowden)

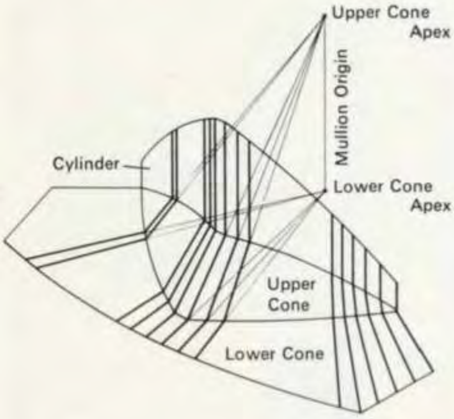


Fig. 9
Glass wall A4: geometry

concrete. This 'continuous strip' as it was called, was bolted on to the rib using the existing holes and was thickened out into corbels to support the mullions (Fig. 8).

No two corbels are identical, nor is the interval between corbels constant and in situ concrete was therefore the most suitable material.

Development of the geometry

The design thus developed into a composite assembly of concrete, steel, bronze and glass. Each material had its own discipline and, just as the geometry of the shells themselves was determined by the precasting requirements for the concrete segments, so the design of the glass walls was dependent on the properties of these four materials. The most important of these material limitations was that of the glass. While glass can be cut into almost any desired shape it would be quite impractical to have sheets of glass that were curved or warped. Each glass sheet, therefore, had to be defined in space by two straight line generators that were coplanar.

The simplest curved surfaces satisfying this requirement are the cylinder, formed by parallel generators, and the cone, formed by connecting a set of points on a curve in space by generators to a common apex.

The A4 glass wall is made up of a cylinder and two cones. Starting from the rib, the surface is a vertical elliptical cylinder, defined by the horizontal projection of the circular rib. Next comes the upper cone which forms a transition surface between the cylinder and the lower

Fig. 11
Glass walls A1 (left) and B1 (right) (Photo: Harry Sowden)

Fig. 12
Entrance foyer (A1)
(Photo: Harry Sowden)

Fig. 13
Glass wall C11 (Photo: Harry Sowden)

cone. The base of the lower cone is related to the geometry of the podium.

The relationship between the shapes of the shells and the podium is mathematically arbitrary and the glass wall is, in effect, an independent geometry that satisfies the boundary conditions of the shell rib at the top and the podium at the bottom. The apex of the upper cone is vertically above that of the lower cone. The axis through them is called the mullion origin and through it pass the vertical planes containing the mullions. This geometry is shown in Fig. 9.

Structural system

The structural system evolved concurrently with the development of the geometry. The mullions were fabricated in two sections, one for the cylinder, the other for the upper and lower cones. The upper sections are bolted to the corbels and tied back at the bottom by struts to the rear wall of the auditorium. The lower sections are bolted to the upper sections and are supported at the lower end by trusses supported, in turn, by V-columns on to the podium (Fig. 10).

Wind load components out of the planes of the mullions are transmitted through the ties between mullions to the centre truss which is formed by the two centre mullions braced diagonally together so as to resist sideways loading. These ties also restrain the mullion chords in compression against buckling laterally.

Glass Walls A1, B1, C11, C12 and Side Walls

These other walls differ considerably in shape from A4 and B4 as not only are the proportions of the shells themselves different but so are the functions of the areas they enclose. Most of the details developed for A4 were used throughout although some simplification was possible, particularly on the side walls, owing to the smaller scale and simpler geometry. Figs. 11 and 12 show the glass walls A1 and B1. Fig. 13 shows the restaurant wall C11, and the view through side wall A5 is shown in Fig. 14.

Research on laminated glass

Although laminated glass has long been used as a windshield material in the aircraft and automobile industries, its widespread use in architectural glazing is a fairly recent development. In windshield applications, it is the impact resistance of the glass which is of prime concern, and this aspect has been extensively studied within the industry. In contrast, it became evident from a preliminary investigation that very little work has been carried out on the response of architectural laminates to normal structural loads, e.g. wind pressure and self-weight loading.

Fig. 14
View through glass wall A5 towards Restaurant
(Photo: Harry Sowden)

Accordingly, a research programme was put into operation with a view to determining the structural behaviour of architectural laminates. It is worth noting, however, that the final dates for establishing the glass specifications and ordering the glass were such that only nine months were available in which to complete the programme.

Bending resistance of laminated section

In order to examine the fundamental behaviour of laminated glass in bending, a series of static loading tests was carried out on a number of beams (approximately 560 mm long, 50 mm wide) of various cross-sectional proportions. These beams consisted of two outer layers of plate or float glass, not necessarily of the same thickness, and a thin plastic interlayer joined in adhesive contact. Two types of commonly available interlayer were tested, one having a smaller plasticizer content than the other; they are designated here as 'hard' and 'soft' interlayers respectively.

As a means of determining the distribution of bending stress throughout the depth of the section, resistance foil strain gauges were attached to each of the four glass surfaces near the beam centre (Fig. 15); this was not entirely straightforward as the two inner gauges had to be affixed prior to the application of heat and pressure which formed part of the laminating process.

During the tests, in which the beams were loaded in four-point bending, measurements were taken of surface strain and central deflection; detailed test results, together with an analytical solution to the problem, are given elsewhere.[3] To summarize, however, the degree of coupling between the two glass layers was found to be chiefly dependent upon the shear modulus of the plastic interlayer. The measured load/deflection curves were sensibly linear, but for beams with a soft interlayer, the bending stiffness was very much lower than that for a monolithic glass beam of the same overall thickness. The effect of this reduction in stiffness on bending stresses is typified by the results given in Fig. 16 for a beam with nominal 10 mm and 6 mm glass layers and a 0.76 mm thick plastic interlayer; here, the maximum tensile stress in the beam with a soft interlayer is 70 per cent greater than the corresponding stress in a similar beam containing a hard interlayer, and 80 per cent greater than the maximum bending stress in a monolithic glass beam of the same overall thickness.

Fig. 15
Laminated glass test beams with strain gauges attached
(Photo: Harry Sowden)

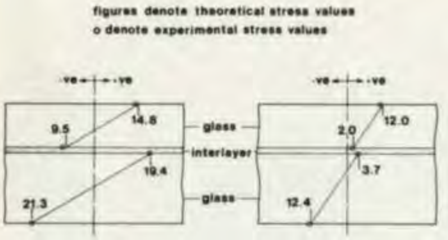


Fig. 16
Theoretical and experimental values of bending stress in laminated glass test beams with soft and hard plastic interlayers (applied load 400 N, span 254 mm, width 50 mm, tensile stresses negative)

Fig. 17
Laminated glass beams under sustained loading

Behaviour of laminated glass under sustained load

Early in the design stage, it became apparent that large areas of glass, e.g. the lower cone areas of A4 and B4, would be glazed in a near-horizontal position, and would therefore be subjected to sustained self-weight loading. It was also clear that the glass in these prominent locations would be exposed to large variations in temperature.

With these problems in mind, experiments were carried out to determine the combined effect of temperature and sustained loading on a number of laminated glass test beams. Creep loading rigs (Fig. 17) were set up in three temperature controlled rooms (at 1.4°, 25° and 49°C), and loads were applied by means of dead weights positioned at the quarter points. The loads were maintained for approximately 80 days, and central beam deflections were measured at intervals throughout this period.

Typical results for a set of six similar beams (nominal 10 mm and 6 mm glass layers, 0.76 mm thick interlayers) at the three test temperatures are shown in Fig. 18. Deflection measurements taken during the initial application of load demonstrate that the bending stiffness of the laminate, and hence the shear modulus of the interlayer, is markedly dependent upon ambient temperature. Furthermore, subsequent deflection measurements show that, except at comparatively low temperatures, the effect of sustained loading is to cause the laminate to deflect as though the glass layers were separated by a material of almost zero shear modulus.

In view of these findings, bending stresses resulting from the self-weight loading of laminated sheets were calculated on the assumption that the glass layers would act independently, without any coupling effect due to the interlayer. Total stresses resulting from a combination of self-weight and wind loading were then determined by superposition.

Fatigue loading of laminated glass

As the effect of wind buffeting against the walls will be to cause individual glass sheets to periodically flex and vibrate, fatigue loading tests were carried out on a number of laminated glass beams of various cross-sectional proportions. The main object of these tests was to ascertain whether or not there was any tendency for delamination to occur.

The loading arrangement is shown in Fig. 19. As the minimum load that could be applied by the fatigue machine far exceeded the strength of the laminated test beams, load was transferred to the glass by means of a purpose-built rig mounted on the machine cross-head. The main component of this rig was a tapered steel beam with adjustable clamps fitted to the tension face; these clamps enabled upward loads to be applied to the glass beams. As before, the laminated test beams were loaded in four-point bending, and the applied loads were such as to induce stresses in the beams of somewhat higher magnitude than those expected in practice. In setting up each test, strain levels in the glass were measured using resistance foil gauges bonded to the outer faces of the beams; during testing, where the frequency of loading was maintained at either 240 or 480 cycles per minute, strain levels were periodically checked by incorporating the gauge signals into an auxiliary circuit to give an oscilloscope display.

The laminated glass responded well to fatigue testing; each beam was subjected to more than one million cycles of stress reversal, but there was no perceptible degree of deterioration in any of the specimens. In addition, static calibration values of strain and deflection obtained after fatigue testing were very similar to the initial measured values.

Full-scale loading tests

The design three-second gust pressure for the glass walls was taken as 1.44 kN/m², which is compatible with that used in the design of the shells. Data obtained from the bending tests on small laminated beams were used to determine the necessary glass sheet thicknesses. Although these data showed that the performance of laminates with a hard interlayer was generally superior to that for laminates with a soft interlayer, there were a number of restrictions imposed by the laminating process itself which meant that, in practice, only the soft interlayer could be used.

In order to provide a check on the full-scale behaviour of the laminates, a series of tests was carried out in which large laminated glass sheets were loaded to failure. The test arrangement, in which four 2.4 m × 1.5 m glass sheets were positioned to form one face of a large suction box, is illustrated in Fig. 20. The glazing of these sheets simulated, as far as possible, the proposed in situ glazing system. Thus the vertical edges were supported by glazing bars, whilst the horizontal edges were supported by pins at each corner. The glazing bars themselves were attached to the tubular steel mullions by means of prototype adjustable fixings. The horizontal glass-to-glass butt joints were sealed with silicone rubber, as were the joints between the glass and the glazing bars. The periphery of the suction box was sealed using a flexible rubber membrane.

The test procedure was simply to evacuate air from within the box and measure plate deflections at discrete pressure levels until failure of one or more of the plates occurred. Several different types of architectural laminates were tested in this way. A detailed account of the results of these tests will be given elsewhere; in general terms, however, there was close agreement between measured and predicted deflections, and when failure did occur the

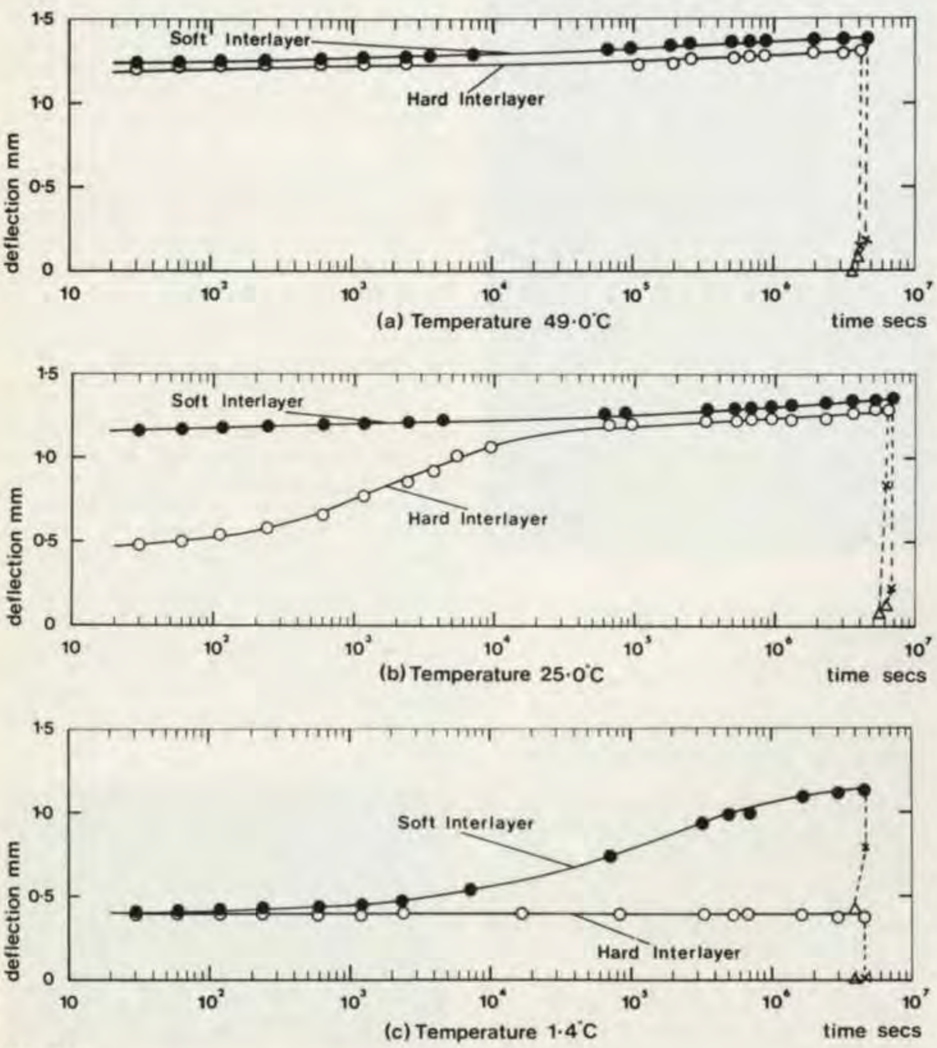


Fig. 18
Experimental creep curves for similar laminated glass beams loaded in four-point bending at various ambient temperatures (applied load 285 N, span 254 mm, width 52 mm; points X relate to immediate unloading, points △ relate to seven days after unloading)

Fig. 19
Arrangement for fatigue loading of laminated glass test beams

Fig. 20
Full-scale loading of laminated glass plates

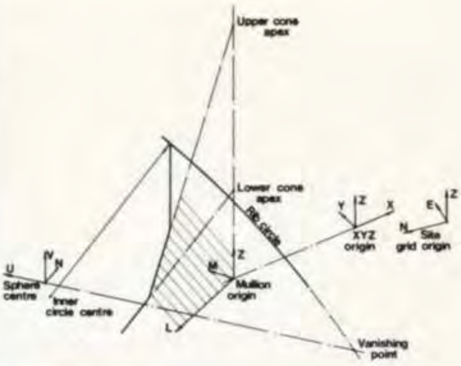


Fig. 21
Co-ordinate systems for A4

fractured plates remained intact, thereby justifying the use of the material itself as a safety glass. Furthermore, the process of actually erecting and sealing the test plates served a useful purpose in drawing attention to a number of handling and construction problems that were likely to be encountered on site.

The geometrical calculations and structural analysis

The geometry

The first step was to define the geometry mathematically. Surveys of the position of the critical ribs showed excellent agreement with the theoretical geometry used in the shell design, the maximum deviation being approximately 25 mm. This degree of accuracy in the construction was remarkable and it meant that it was possible to work to the theoretical geometry. The connections between mullions and ribs could be detailed to take up the discrepancies.

The podium geometry had also been defined for the design of the precast cladding. The consistent geometrical discipline that had been maintained in both shells and podium meant that only minor refinements in the terminology were necessary to completely formulate the problem mathematically.

The shell and podium geometries involved numerous co-ordinate systems. Additional systems relating to the geometry of the walls themselves were also established and the main systems used for A4 are shown in Fig. 21.

Each shell is principally defined by two points: The sphere centre (*XYZ* co-ordinates). This is the centre of the spherical outer tiled surface; The vanishing point (*XYZ* co-ordinates). This point corresponds to the north pole of the sphere.

Each rib is defined by a plane about which it is symmetrical and each rib plane passes through the axis joining the sphere centre and vanishing point (i.e. at constant longitude, like the planes separating segments of an orange).

The inside or soffit of each rib is a cylinder, the axis of which is normal to the rib plane, but is offset from the sphere centre so that each rib increases in depth from a minimum at the pedestal to a maximum at the ridge. The point where the axis of the soffit cylinder passes through the rib plane is called the inner circle centre and is unique for each rib. Each rib plane is defined by its normal vector which is related to the *XYZ* system by direction cosines. Each point on the cross-section of rib and continuous strip follows a circle in space which can be defined by *N* and *R* co-ordinates, the offset *N* from the rib plane and the radius *R*.

The podium shape is defined in terms of the site grid (*ENZ*) and the various key points were readily converted into *XYZ* values.

The usual sequence was to solve points in *XYZ* co-ordinates, then transform them into the other systems. From the computer output, overall layout drawings and framing plans were plotted in *XYZ* co-ordinates, but the other co-ordinate systems were generally more useful for plotting critical details to larger scales. For example, true views of the continuous strip could be plotted in *NUV* co-ordinates while *MLZ* co-ordinates were more suitable for the mullions. A procedure of numerical analysis and graphical appraisal evolved.

The form of the calculations is best described by an example. The following is the derivation of points on a typical corbel shown in Figs. 8 and 22. The mullion plane has already been calculated and this determines the face of the corbel. The steel mullion hangs from the bolt which projects from the face of the corbel through Point 1.

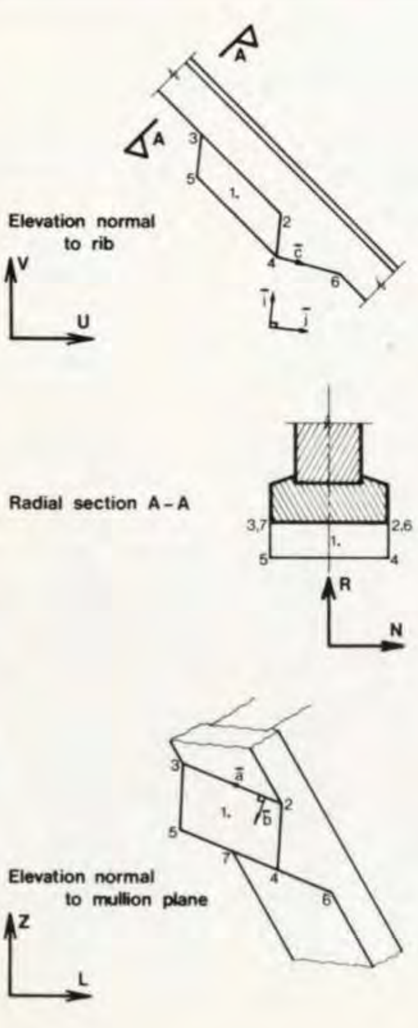


Fig. 22
Views of section of continuous strip with corbel

Definitions

p_c position vector (i.e. *XYZ* co-ordinates) of the inner circle centre of the rib

p_{mo} position vector of the mullion origin

$\bar{n}$ the unit normal vector to the rib plane

$\bar{m}$ the unit normal vector to the mullion plane

N_1R_1 the N and R co-ordinates of the circle through point 1.

Define $i=\bar{m}\wedge\bar{n}$

Normalize i to form $\bar{i}$, i.e. the unit vector

Define $\bar{j}=\bar{i}\wedge\bar{n}$

$\bar{n}$, $\bar{i}$ and $\bar{j}$ form an orthagonal set of three vectors, i and j lying in the plane of the rib and the position vector of point 1 can be expressed as follows:

$$p_1=p_c+N_1\bar{n}+I\bar{i}+J\bar{j} \qquad 1$$

where I and J are unknown scalars.

The condition that the point lies on a cylinder radius R_1 is that:

$$R_1^2=I^2+J^2 \qquad 2$$

The condition that the point lies on the mullion plane is that:

$$(p_1-p_{mo}).\bar{m}=0 \qquad 3$$

Substituting 1 in 3

$$(p_c-p_{mo}+N_1\bar{n}+I\bar{i}+J\bar{j}).\bar{m}=0$$

noting that $\bar{i}.\bar{m}=0$

then $J=(p_{mo}-p_c-N_1\bar{n}).\bar{m}/\bar{j}.\bar{m}$

while $I=\sqrt{(R_1^2-J^2)}$

thus I and J are evaluated and the position p_1 is found from Equation 1.

The procedure is repeated for points 2 and 3 with the corresponding values of N and R.

The points 4 and 5 are defined and calculated as follows:

The edge 4–5 is made parallel to the line joining points 2 and 3. The perpendicular distance from the bolt to this edge is fixed at a constant dimension B. The unit vector $\bar{a}$ is found between points 2 and 3.

Defining $\bar{b}=\bar{m}\wedge\bar{a}$

then the position of point 4 can be expressed by

$$p_4=p_1+B\bar{b}+A\bar{a} \qquad 4$$

where A is unknown.

If the N co-ordinate of point 4 has the fixed value N_4.

Then $(p_4-p_c).\bar{n}=N_4$ 5

Substituting Equation 4 in 5 and noting that $(p_1-p_c).\bar{n}=N_1$ then $A=(N_4-N_1-B\bar{b}.\bar{n})/\bar{a}.\bar{n}$ the scalar A is evaluated and p_4 found from Equation 4.

The process is repeated for point 5.

The soffit of the corbel is made perpendicular to the corbel face and is fixed by the edge 4–5.

The normal vector to the soffit plane is therefore $\bar{b}$. Points 6 and 7 are the intersection of the corners of the continuous strip with this plane.

The vector along the edge 4–6 is given by $c=\bar{n}\wedge\bar{b}$.

The position vector of point 6 can be expressed by

$$p_6 = p_4 + C\bar{c} \qquad 6$$

where C is an unknown scalar.

The condition that point 6 lies on a circle radius R_6 is given by

$$|p_4 - p_c - N_4\bar{n} + C\bar{c}| = R_6 \qquad 7$$

which reduces to a quadratic in C

$$C^2 + 2C(p_4 - p_c)\,.\,\bar{c} + R_4{}^2 - R_6{}^2 = 0$$

for which C is calculated and the position of point 6 is found from Equation 6.

These formulae are readily translated into Fortran as follows:

P1 position vector array of point 1 containing co-ordinates *XYZ*, *NUV*, *RML*.

P2, *P3* etc., similar arrays for points 2, 3, etc.

PC, *PMO* position vector arrays p_c p_{mo} containing co-ordinates *XYZ*.

PS position vector of the sphere centre (origin of *NUV* co-ordinate system).

VN, *VM*, *VI*, etc., arrays containing the three components of vectors $\bar{m}$, $\bar{n}$, $\bar{l}$, etc. *SJ*, *SB* scalar quantities *J*, *B*, etc.

Subroutines were used for the basic vector operations:

```
SUBROUTINE VPROD (VA, VB, VC) (Vector
               Product)
DIMENSION   VA (3), VB(3), VC(3)
VC(1) = VA(2)*VB(3)-VA(3)*VB(2)
VC(2) = -VA(1)*VB(3)+VA(3)*VB(1)
VC(3) = VA(1)*VB(2)-VA(2)*VB(1)
RETURN
END
SUBROUTINE VUNIT(VA) (Vector Normal-
izing)
DIMENSION   VA(3)
A     = SQRT(VA(1)**2+VA(2)**2+
        VA(3)**2)
VA(1) = VA(1)/A
VA(2) = VA(2)/A
VA(3) = VA(3)/A
RETURN
END
SUBROUTINE VADD (VA, S, VB, VC) (Vec-
tor Addition or Subtraction)
DIMENSION   VA(3), VB(3), VC(3)
VC(1) = VA(1)+S*VB(1)
VC(2) = VA(2)+S*VB(2)
VC(3) = VA(3)+S*VB(3)
RETURN
END
FUNCTION    SPROD (VA, VB) (Scalar Pro-
duct)
DIMENSION   VA(3), VB(3)
SPROD=VA(1)*VB(1)+VA(2)*VB(2)+
       VA(3)*VB(3)
RETURN
END
```

The routine for calculating point 1 is then as follows:

```
CALL  VPROD (VM, VN, VI)
CALL  VUNIT (VI)
CALL  VPROD (VI, VN, VJ)
CALL  VADD (PC, P1(4), VN, P1)
CALL  VADD (PMO, -1.0, P1, V)
SJ    =SPROD (V, VM)/SPROD (VJ, VM)
CALL  VADD(P1,SQRT(P1(7)**2-SJ**2),
      VI, P1)
CALL  VADD (P1, SJ, VJ, P1)
```

The other co-ordinates of point 1 are then found by simple transformation.

```
CALL  VADD (P1, -1.0, PS, V)
P1(5) =SPROD (V, VU)
P1(6) =SPROD (V, VV)
CALL  VADD (P1, -1.0, PMO, V)
P1(9) =SPROD (V, VL)
```

The routine for points 4 and 6 is:

```
CALL  VADD (P2, -1.0, P3, VA)
CALL  VUNIT (VA)
CALL  VPROD (VM, VA, VB)
CALL  VADD  (P1, (P4(4) - P1(4) -
      SB*SPROD (VB, VN))/SPROD (VA,
      VN), VA, P4)
CALL  VADD (P4, SB, VB, P4)
CALL  VPROD (VN, VB, VC)
CALL  VUNIT (VC)
CALL  VADD (P4, -1.0, PC, V)
P4(7)=SQRT (V(1)**2+V(2)**2+
      V(3)**2-P4(4)**2)
B     =SPROD (V, VC)
CALL  VADD (P4, -B+SQRT (B**2-
      P4(7)**2+P6(7)**2), VC, P6)
```


These routines were, of course, only a small part of the complete program for the continuous strip. Other points, critical points on the steel mullion and glass had also to be calculated and checked.

From the co-ordinates, dimensions necessary for the prefabrication of the formwork were calculated. These were output in the form of schedules. Bar bending dimensions for the reinforcement were also calculated and scheduled.

Generally two programs were written for each wall, one for the continuous strip and corbels and a separate program for the rest of the wall. This covered the steel mullions, the ties between the mullions, the glazing bars and fixings, the glass geometry, the arrangement and dimensions of glass sheets and the structural analysis of the mullion-tie system.

The resulting programs were, in effect, mathematical models, accurate yet highly flexible. The units used were feet and dimensions and co-ordinates were printed to three decimal places. Typical pages of computer output are shown in Fig. 23. From the same output the whole glass wall could be drawn to be a small scale or alternatively several views of a critical detail drawn full size. By altering the data, the effects of a change at one point could be checked at other critical points. Alternatively the geometrical definitions could be amended by re-programming the different routines.

The details were developed in parallel with the computer programs and the geometry. This was essential because the overall geometry determined the size of members and the form of the connections while these in turn influenced the overall geometry.

The necessary flexibility was easily achieved in the computer programs by setting up an array of constants into which these dimensions could be read as data. As the design was developed, new constants were built into the program while others became redundant and, to save time in re-programming, even the alphanumeric headings in the data output were read in as data themselves. By the time the A4 programs were complete, there were 60 such constants, ranging from the co-ordinates of the mullion origin and cone apices to the width of the silicone rubber joints and the diameter of the corbel reinforcing bars.

During the development of the design, it was the co-ordinates of the different points that were most useful, but the end result of the whole process was working drawings and dimensions. Altogether over 30,000 dimensions were supplied to the contractor. Some were transferred to the working drawings but most were issued in schedule form; here it was generally possible to design the format so that it could be photocopied and issued directly.

The program for A4 was developed over a period of a year and was run twice in its final form, once for A4 and once for B4.

Structural analysis

The structural analysis was in principle quite simple. The mullions themselves are statically determinate, but the three-dimensional shape of the whole wall was such that the calculations would have been tedious. Sub-routines

DATE 15/03/71

TITLE RESTAURANT C11 7288/71 C11 60

DATA	X	Y	Z
CYLINDER CENTRE	-94.817	158.371	-103.359
NORMAL VECTOR	0.892845	-0.357057	-0.274478

CONSTANTS

1	MULLION ORIGIN X	-160.000000	23	DEAD LOAD LINEAR LBS/FT	50.000000
2	CONE APEX Z	88.910000	24	DEAD LOAD/AREA LBS/SQ.FT	10.000000
3	MULLION 1 PT.3 X	-75.236000	25	WIND LOAD/AREA LBS/SQ.FT	30.000000
4	MULLION 1 PT.3 Y	-3.054000	26	PT.3 UDL LBS/FT	90.000000
5	Z LEVEL PT.3	41.410000	27	TIE CONNECTION LENGTH CYLINDER	1.313000
6	SPIRAL COEFFICIENT	0.971188	28	TIE CONNECTION LENGTH CONE	1.146000
7	DIMENSION 5 MULLION 2	6.386000	29	TIE CONNECTION LENGTH HORIZ ARCH	1.813000
8	NO. OF MULLIONS	12.000000	30	GLASS CUTTING FRAME DIMENSION	13.000000

MULLION NO. 3

MULLION PT.	X	Y	Z	L	A	CHORD	ZONE
1	-106.149	-9.187	77.788	54.629			
2	-106.149	-9.187	55.825	54.629			
3	-82.687	-13.190	41.464	78.430			
4	-85.549	-12.702	41.464	75.527			
5	-87.919	-12.297	41.464	73.122			
GLASS POINTS							
1	-105.575	-9.285	77.594	55.212			
2	-105.575	-9.285	56.159	55.212			
3	-82.316	-13.253	41.869	78.807			
4	-82.358	-13.470	41.110	71.651			
5	-86.116	-10.267	34.035	66.713			
TIE POINTS	X	Y	Z	L	A	CHORD	ZONE
A01	-106.149	-9.187	73.662	54.629	17.837	OUTER	CYLINDER
A02	-106.149	-9.187	64.152	54.629	8.327	OUTER	CYLINDER
A03	-106.149	-9.187	56.075	54.629	0.250	OUTER	CYLINDER
C0	-100.513	-10.149	52.375	60.346	6.678	OUTER	CONE
D0	-93.143	-11.406	47.864	67.823	15.410	OUTER	CONE
E0	-84.897	-12.813	42.817	76.188	25.180	OUTER	CONE

Fig. 23
Typical computer print-out

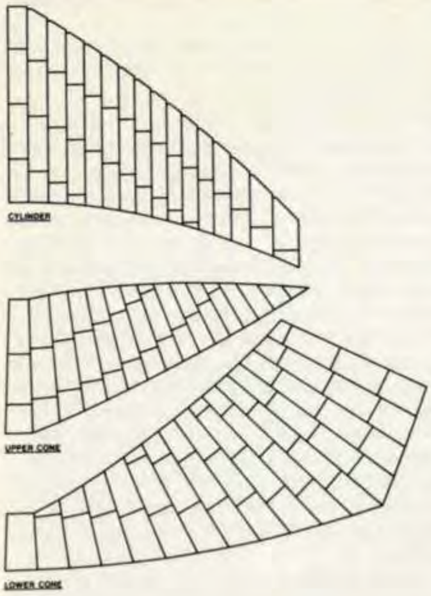


Fig. 24
Glass wall A4: developed surfaces

were therefore written into the geometrical programs to calculate the reactions and tie forces due to dead and wind loads. These results then became data for the analysis of the indeterminate parts of the structure using standard frame analysis programs.

Accurate prediction of deflections was particularly important for the restaurant walls. The absence of any structure connecting to the podium, providing uninterrupted views through the view window, was achieved at the cost of significant vertical deflections of the structure under dead load.

This problem was overcome by supporting the steelwork during erection, in positions which would allow for the deflections that would occur later, until all the structural connections were complete. The steelwork dimensions were calculated so that the structure would fit together in this undeflected shape. The glass dimensions were based on the theoretical geometry that the wall would take up after deflection.

Dimensioning of glass sheets

Once the steel structure and glazing bars had been erected, it would have been possible to measure the openings and cut each sheet of glass to suit. This approach, however, was rejected, except for special cases, as it would have been too time-consuming.

The alternative was therefore adopted of cutting the glass sheets to theoretical calculated dimensions wherever possible. It was decided, in view of the delivery period for the glass from France, which was about four months, that the glass would be supplied as slightly oversize rectangular blanks and cut to final shape on site.

When the layout of the structure and position of the glass had been established, developed views of the glass surfaces were drawn from the computer output, giving a true view of each glass plane and the lines of the glazing bars (Fig. 24). The architect then set out the general pattern of glass sheets. This pattern was then programmed and the blank sizes calculated and issued to the contractor so that the glass could be ordered.

The final dimensions were calculated when all the glazing details had been finalized. In general, the shapes were irregular quadrilaterals and the problem of dimensioning these was solved by using a standard dimensioning frame. A table was set up to which was fixed a rectangular frame, the sides of which were graduated in decimals of a foot. The glass was placed in the frame and dimensions measured up the sides of the rectangle. Using a straight edge, the four sides of the required shape could then be drawn. Once the shape had been set out, the lengths of the sides and diagonals could then be checked. A typical glass sheet schedule is shown in Fig. 25.

Method of shaping laminated glass sheets

One of the major problems associated with the construction of the glass walls was how best to shape the individual glass sheets prior to erection, particularly as hundreds of different sized sheets were required; reference to Fig. 24, for example, shows that almost every sheet of glass on A4 is a different shape.

In practice, there are two possible methods of shaping laminated glass, namely by sawing, using a diamond-impregnated blade, or by the traditional process of cutting and breaking. In this latter technique, both glass layers are scribed and broken in turn, followed by the application of heat along the line of fracture. With the plastic interlayer thus softened, the two sections are pulled apart and finally separated using a thin blade. But although this method is well tried, it is slow and laborious when dealing with relatively thick sections of laminated glass; it also results in stepped edges which require a good deal of subsequent machining and finishing work.

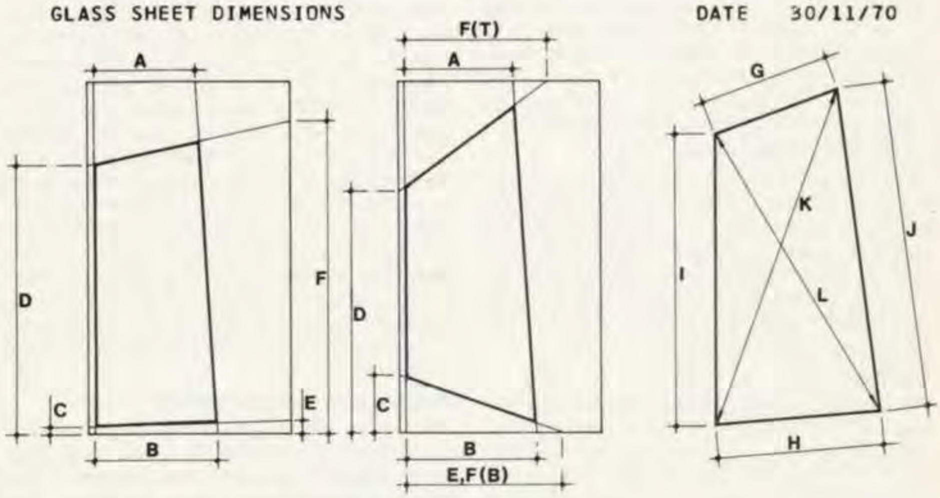
Sawing, on the other hand, can give an edge which is straight and flat and which only needs arrising to render it acceptable for glazing. However, it was envisaged that difficulties might arise in obtaining a sufficiently high speed of travel to be compatible with construction programme requirements. An enormous amount of glass had to be sawn, and therefore production rates were important. But as the linear speed of sawing increases, so does the heat which is generated, and if overheating of the laminate does occur, it produces a number of undesirable effects, notably firing of the plastic interlayer and excessive spalling of glass.

Nevertheless, the basic decision was made to adopt the sawing method, and two purpose-built sawing machines were designed and developed by specialist sub-contractors. Investigations were also directed towards finding the most suitable type of saw blade. Eventually a factory was set up on site to deal solely with the processing and handling of the glass prior to erection.

One of the sawing machines is illustrated in Fig. 26. The two end-frames were connected

OVE ARUP AND PARTNERS
SYDNEY OPERA HOUSE STAGE III
GLASS WALL A4
GLASS SHEET DIMENSIONS

SCHEDULE NO.
1112/7280/T1
PAGE 53 OF 132
DATE 30/11/70



SHEET NO A4/206 U/CONE

SET OUT DIMENSIONS

	WEST SIDE	EAST SIDE
A	5.206	5.197
B	5.925	5.916
C	0.065	1.857
D	7.433	7.269
E	2.715	B 6.252
F	7.640	7.476

CHECK DIMENSIONS

G	5.507	I	7.335	K	9.294
H	6.158	J	5.444	L	7.850

ALL DIMENSIONS IN FEET. REFER DRG. NO. 1112/7473

Fig. 25
Typical glass sheet schedule

Fig. 26
Sawing machine (Photo: Harry Sowden)

by a heavy cross-member of box-type construction which accommodated the movable head. Pneumatically operated castors were built-in to the working table to facilitate easy handling of the glass; during sawing, with the castors retracted, the glass could be firmly held by suction pads. The glass was sawn in a single pass along the prescribed line and then, by pneumatically reversing the chuck, the edges were arrised on the return pass by abrasive wheels. This procedure was then repeated for the remaining edges.

The initial problems of overheating and vibration were overcome, and the sawing operation proved to be entirely successful in producing glass edges of the high standard required. After some experimentation, linear feed rates of up to 0.9 m/minute were attained, but the rate subsequently adopted for general production purposes was 0.5 m/minute.

Erection procedure

The shuttering and reinforcement to the continuous strip, prefabricated in short lengths to the calculated dimensions, were lifted up and attached to the rib. Survey stations on each mullion plane had been established, enabling the formwork to be moved up or down the rib until the face of the corbel lay in the mullion plane. The section of concrete was then cast. When the continuous strip was complete, each corbel bolt was surveyed and the co-ordinates fed into the computer program. This calculated the position on the steel mullion where the hole for the corbel bolt should be drilled.

The prefabricated steelwork could then be erected and would automatically be located in its theoretical position, independent of the deviations of the shell rib itself.

The steel ties between the mullions were adjustable in length to allow for tolerances. However, such was the quality and accuracy of the steel fabrication that the full range of adjustment was generally not required. On the other hand it was invaluable as a means of aligning the mullions into the surveyed planes as they were quite flexible in the lateral direction.

After the steelwork had been painted, the fixing brackets and glazing bars were attached. Both mullions and glazing bars were shop drilled for the fixings. The glazing bars were aligned using plywood templates made to dimensions calculated in the same way as the dimensions of the glass sheets, and check surveys were made on specified points. The edges of the glass sheets adjacent to the continuous strip, and around the corbels, were templated to allow for deviations of the shell rib from its theoretical position.

The glazing was carried out from the top downwards from working platforms supported on scaffolding. The laminated glass sheets, having been sawn to shape on site, were hoisted on to the platform, normally in groups of three or four. On the platform itself, individual sheets were moved by means of a purpose-built mobile crane with suction lifting equipment. Special jigs were clamped on to the glazing bars in order to provide temporary support for the glass and to enable the glass sheet to be accurately aligned. Each jig incorporated a drill attachment which was then used to drill holes in the glazing bar flanges to take the push-fit glass support pins. With the glass in its correct position and held against the glazing bars by means of temporary clamps, the joints were sealed with silicone rubber, as described in the following section.

Sealing and waterproofing

With much of the glass inclined to the horizontal and situated directly above areas used by the public, it was particularly important that the walls should be completely watertight. Use of the best available type of sealant was essential, and silicone rubber (translucent type 3B) was selected for this purpose.

The choice of silicone rubber was really dictated by the presence of the horizontal glass-to-glass butt joints. These joints are directly exposed to the atmosphere and silicone rubber, besides having an excellent adhesion to glass, has a high resistance to ultra-violet radiation and other weathering agencies. Furthermore, its elastic properties ensure that it remains permanently in place on the inclined glass surfaces; most other sealant materials exhibit time-dependent flow properties which would cause them to sag and perhaps disappear from the joint altogether.

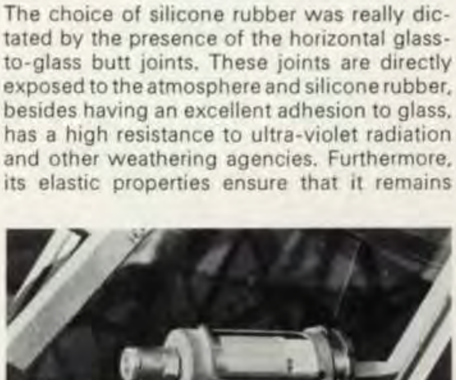


Fig. 27
Application of silicone rubber seal (Photo: Harry Sowden)

Silicone rubber, being a one-part sealant, is relatively easy to apply; compressed air guns were used on site, as illustrated in Fig. 27. However, its use as a construction sealant poses difficulties in that the substrates have to be cleaned and prepared to quite stringent standards. In the present case, for example, the sawn edges of laminated glass had to be provided with protective covers during handling and storage, and carefully cleaned just prior to glazing. In addition, the glazing bar surfaces had to be abraded, solvent cleaned and primed prior to application of the silicone rubber. Some experimentation was even required in selecting the most appropriate primer; it had to be insensitive to surface moisture and was required to resist attack from the acetic acid which is given off by the silicone rubber during curing.

On the main glass surfaces, the primary seal is provided by the horizontal glass-to-glass butt joints and by the vertical joints between the glass and the web of the glazing bar T-section. As a second line of defence, the glazing bar cover strip was assembled in such a way as to effect a form of mechanical seal. In this arrangement, the cover strip was left loose during application of the outer silicone rubber seal; when the rubber had cured, the cover strip was screwed down on to the web of the T-section, thereby causing compression of the rubber and giving a continuous external gasket along the joints in the vertical plane. At joint locations with no cover strip, such as those near the lines of intersection between the principal glass surfaces, the silicone was made to bridge between adjacent glass edges by cutting back the web of the glazing bar.

The silicone rubber should maintain a durable and watertight seal for a great many years. Some future maintenance and repair work is inevitable, but evidence from both accelerated weatherometer tests and outdoor exposure tests strongly suggests that the useful life of silicone rubber is well in excess of 20 years.

Acknowledgements

The architects for Stage 3 of the Opera House were Hall, Todd and Littlemore, and the structural engineers were Ove Arup & Partners. The main contractor was M. R. Hornibrook Pty Ltd. and the subcontractors for the glass walls were J. W. Broomhead Pty Ltd (steelwork), Permasteel Pty Ltd (metalwork), Hawker de Havilland (Aust.) Pty Ltd (fixings), Quick-Steel Engineering Pty Ltd (sawing machinery and glass handling equipment), and VASOB Glass Pty Ltd (glazing). The bronze extrusions were supplied by Austral Bronze Crane Copper Pty Ltd, the glass by Boussois Souchon Neuvesel/ Société Industrielle Triplex, and the sealant by Rhône-Poulenc.

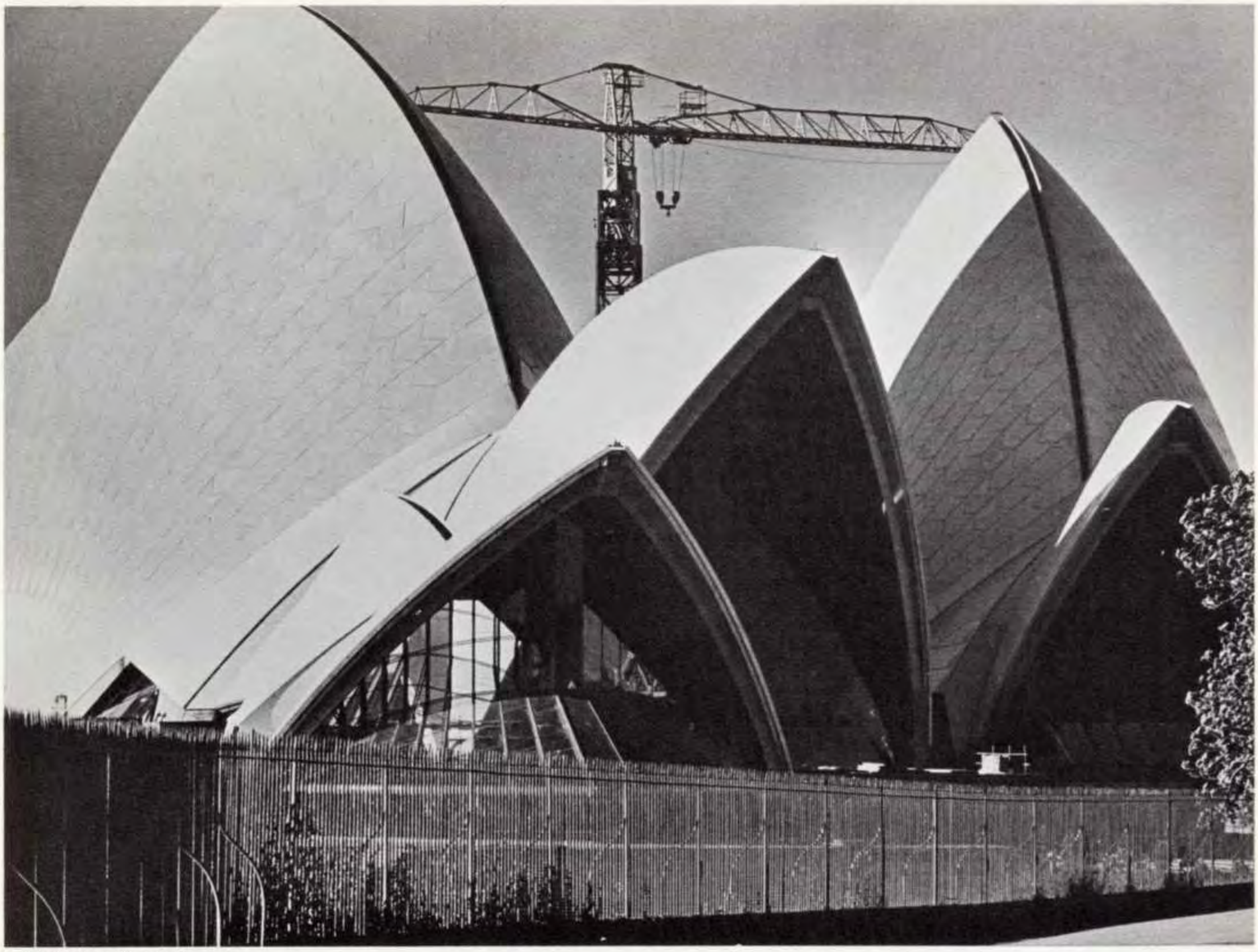
The experimental programme was financed by the Government of New South Wales. The small-scale beam loading tests were carried out at the Materials Testing Laboratory of the Public Works Department, Sydney. The sustained-load tests were carried out at the CSIRO Division of Food Preservation, Sydney, and the fatigue tests at the Civil Engineering Department, Sydney University. The full-scale plate loading tests took place at the Commonwealth Experimental Building Station, Sydney.

References

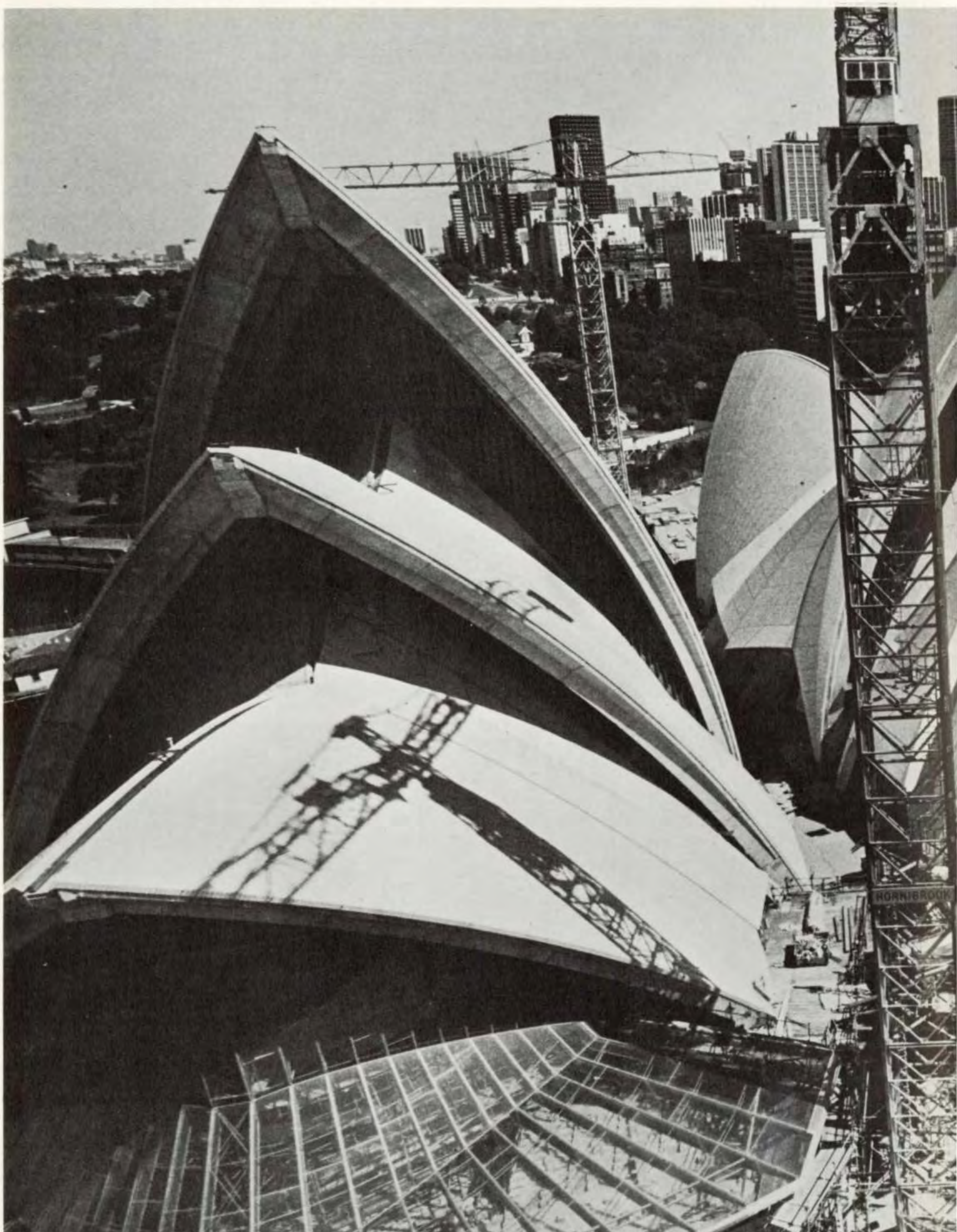
(1) ARUP, O. N. *and* ZUNZ, G. J. Sydney Opera House. *The Structural Engineer*, **47** (3), pp. 99–132, 1969.

(2) SOWDEN, H., *Editor*. Sydney Opera House Glass Walls. Sands, Sydney, 1972.

(3) HOOPER, J. A. On the bending of architectural laminated glass. *International Journal of Mechanical Sciences*, **15** (4), pp. 309–323, 1973.



(Photos: Harry Sowden)



 (Photo: Harry Sowden)

Grouting prestressing ducts

John Nutt

This paper was given to the Australian Prestressed Concrete Group in August 1967.

Introduction

The purpose of grouting prestressing ducts is twofold – to prevent corrosion of the tendon and to provide bond between the tendon and the concrete. The properties of grout which determine the rust-preventing properties are well known and even a grout of fairly poor composition is usually sufficient to prevent oxygen and water penetrating to the steel. Under normal working loads no cracks develop in the prestressed concrete, and in general only very low shear stresses occur between the tendon and the concrete. Only when the working load is exceeded and cracks occur do fairly high bond stresses develop. Thus the task performed by bond is very much smaller in the case of prestressed concrete than in reinforced concrete. Furthermore, if unforeseen corrosion takes place and the tendon rusts at one section, the bond between the tendon and the concrete will ensure that the prestress force of that tendon is not lost over the entire length of the tendon even if corrosion should result in the failure of a tendon.

While the purpose for grout is universally agreed upon, the desirable procedures of grouting are the subject of greatly differing opinions and the techniques to be used in any particular case are well debated.

The intention of this paper is to outline a number of techniques in grouting and grout investigations developed at the Sydney Opera House, some of which we think are original and of merit, and to discuss these techniques in relation to the problems they overcame. It should be stated now that such developments as did take place were the result of a co-operative effort on the part of the contractor, M. R. Hornibrook and the consulting engineers, Ove Arup & Partners. Our problems were unique and we had recourse to discussions with many people in the civil engineering field, in the chemical industry and in a number of associated spheres of activity. We found a high degree of willingness to help and to impart knowledge. In order to place the subject in perspective, it is necessary to explain briefly some aspects of the roof structure of the Sydney Opera House.

All longitudinal ducts (Fig. 21, p.11, and Fig. 1, right) were formed in the concrete by inflated ductube. Wire loops cast into the concrete around the ductube reduced the diameter by 3 mm at 305 mm centres along the length. The steel/steel bearing between the tendon and the loops was introduced to reduce friction.

The erection strands were anchored at the end of each segment, at a temporary anchorage which provided a considerable restriction in the duct, leaving only a 3 mm clear annulus around the 35 mm duct.

In addition to the ducts, the flanges of concrete also carried other items which were subsequently shown to have had an effect on the grouting. These were:

1 Transverse holes passing between the ducts to take lateral bolts and lateral prestressing cables. These were sheathed either with plastic linings to which a plastic flange had been added, or with corrugated metal tubing.
2 Adjacent bolting pockets
3 Adjacent internal voids in solid segments
4 A longitudinal lead groove cast into the flange to take rib/rib flashing
5 Bronze tile lid bolts passing between the ducts.

The variety and extent of these 'secondary' members in a concrete section already heavily congested with reinforcement and prestressing ducts resulted in serious difficulties in grouting. The longitudinal rib ducts form the most important group of ducts for the roof structure and the subject matter of this talk is orientated around them. However, there are many others. In all there are 4100 individual ducts with a total length of about 110 km.

Original grout specification

The original grout specification had been used with reasonable success on numerous structures with near-horizontal cables – it contained similar clauses to the recommendations embodied in the 1961 Prestressed Concrete Development Group Recommendation. The salient features were:

1 Maximum water/cement ratio to be 0.45, preferably 0.4
2 Specified strength – 17.2 N/mm² at seven days for 100 mm cubes
3 Bleed – maximum of 3 per cent after three hours
4 Pumping pressure available – 0.7 N/mm² minimum
5 Technique – Ducts to be flushed with water prior to grouting
Grouting to take place from lowest point
Grout to run freely from intermediate and top vents prior to blocking off
Ducts to be topped off after 24 hours.

Grouting technique

A number of trials and experiments were instituted on the first few ducts to decide on a satisfactory system. Because of the magnitude of the grouting and the manner in which the ducts were spread about the site, the contractor decided to centralize the grout mixing plant and circulate the grout around the site by leading off from a closed loop.

The ducts were first water tested and the leaks blocked either by caulking or by using an epoxy-resin mortar. Grout connection was made to the duct at the bottom anchorage through the 6 mm diameter hole in the lower anchorage and grout pumped in at the rate of about 14 litre/min. Grout bleed holes cast in at the ends of each 4.6 m segment and taking the form of 6 mm diameter plastic tubes leading from the duct to the outer surface of the shell, provided an opportunity to follow the progress of the head of this grout. These bleed holes were blocked using wooden plugs after grout had flowed freely from them. Grouting was continued until a satisfactory flow of grout came from the bleed hole in the top anchor plate which was always the highest point of the duct. Many of the top anchors were buried in concrete soon after stressing and this top bleed hole was sometimes 1.2–1.8 m long as a result. The flow was then turned off at the input valve and the grout allowed to set. After settlement had taken place, the column of grout was topped up by trickling additional grout into the duct from the top grout hole. The topping-up procedure was subsequently modified after a number of ducts had been grouted. Instead, a standpipe was connected to the top anchor grout hole and a plastic bag to act as a reservoir attached to the top of this standpipe. Grout would be pumped up through the duct into the standpipe and reservoir and would flow back into the duct as bleeding and shrinkage took place. It was considered that this would leave the duct fully grouted. Both methods were adaptions of techniques which have been successfully used elsewhere.

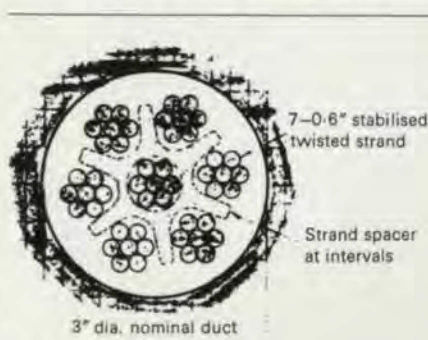
Initially, the grout used was a neat cement grout 0.5 water/cement ratio with a wax-emulsion additive to act as a fluidity aid. This mix was subsequently changed to a water/cement ratio of 0.43 with a calcium liquosulphonate additive.

Difficulties

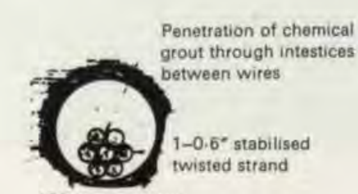
The contractors' task, like most things on the Opera House, was anything but easy; he was called upon to grout an exceedingly complex network of ducts in inaccessible positions and under hazardous conditions – men would frequently be working off Bosun's chairs. Many difficulties were encountered, the chief of which were as follows:

1 Frequent blockage occurred in the grout circulation system necessitating a grout of high fluidity which frequently tested outside specification limit for bleed.
2 High pressures were involved – pumping to a height of 45.7 m required a pressure head of 1 N/mm² at the bottom and, together with the head loss due to pipe friction, frequently required pressures of 2.4 N/mm² to be applied at the pump.
3 Leaks under water testing were frequently hard to detect. Many would occur on the inside of the shells where access was difficult and dangerous. This would frequently mean that a duct would have to be flushed out with water before the leak could be stopped. Some ducts had to be flushed out several times before all leaks could be eliminated.
4 Conditions changed from duct to duct and rib to rib. Techniques developed for one duct would be inadequate for another.
5 Checking the efficacy of the grouting was impossible for many ducts. Established techniques of examination would not work.
6 Many of the ducts were found to be interconnected. All ducts so found had to be grouted simultaneously.

It may appear from a casual reading of all these problems that the quality of concrete composition was suspect – anybody who has been onto the construction site will verify that this is certainly not the case, but these difficulties led to the doubt that a number of ducts were not effectively filled and it was suspected that the techniques used for topping-up the cables after bleed were not fully effective. A simulated trial carried out on an 18.3 m length of 25 mm plastic pipe increased these fears when it showed void formation in the column of grout under restrictions. A further check on a number of ducts showed that large top voids were formed and that the initial trials on the use of the bleed bag technique for topping-up were misleading and



2 MULTISTRAND DUCT



1 SINGLE STRAND DUCT

Fig. 1
Cable and duct arrangements

no reliance could be placed upon its use. The possibility of intermediate void formation caused great concern and an experimental and investigation programme was put in hand to determine the extent of the trouble. Simultaneously enquiries were put out in this country, in the UK and in Europe to determine previous experience in such void formation.

Experimental programme

It was hoped from the experiments to:

1 Determine the cause of void formation
2 Determine the extent of the voids in the already grouted cables
3 Develop techniques and grouts to make good the already grouted ducts, if necessary
4 Devise techniques and grouts to prevent the occurrence taking place in future duct grouting operations.

Cause of void formation

Tests were undertaken on model ducts to observe the column of grout over the entire length of grout:

(a) *Tests on 25 mm diameter clear plastic tubes*
These ducts, between 18.3 m and 30.5 m long, each containing one unstressed 25 mm diameter strand, were erected on the outside of one of the shells to simulate the ducts in their nearly-vertical orientations. As soon as the column of grout ceased to be in motion, separation started. Water and trapped air bubbles moved to the upper side of the duct and moved quickly up that side to reach to top of the grout column. Grout settled very slowly in the rest of the duct. The entire column (grout and water) settled about 2 per cent and bleed water collected to a depth of about 5 per cent at the top. The bleed recorded in the 300 mm laboratory tests was about 2 per cent.

Below a number of the restrictions it was noticed that separation of the grout column took place – the stiffening grout arching across the duct allowing water to collect immediately below the restrictions. This separation never took place in the lower portion of the duct.

(b) *Tests on 75 mm diameter plastic tubes containing seven 15 mm cables*
Two 12.2 m long plastic ducts were attached to the side of the shell and grouted in a similar manner as before. Grout bleed varied between 4 per cent and 7 per cent and the filling speed of the duct was 4.6 m/min. Bleed water collected at the top of the grout, but no intermediate voids were found.

(c) *Tests on 75 mm model concrete ducts*
To simulate the effect of wall roughness and absorption ability, five ducts were constructed in 125 mm galvanized-steel tubes by concreting the annulus between the tube and a 75 mm ductube within it.

When the ductube was withdrawn, a tendon of seven 15 mm strands was threaded through and the duct grouted in the normal manner. Breaking open the tubes revealed the formation of a top void but no intermediate voids (Fig. 2).

As a result of a number of these tests the following conclusions were drawn:

The bleeds recorded in long vertical ducts were considerably in excess of that measured in 300 mm long laboratory tests. This was subsequently confirmed in numerous other tests on model ducts. Frequently this bleed was three times greater than that measured on smaller samples.

In the 35 mm ducts, intermediate voids could occur below restrictions in plastic tubes.

The amount of air trapped below the surface of the grout in a duct was affected by the speed of grouting. This air would eventually rise to the surface to contribute to the length of the top void.

The standpipe and topping-up technique were not effective in preventing the formation of the top void. After some stiffening of the grout, blockage could occur in the 6 mm grout hole of the top anchorage. A void would form in the duct and would remain unfilled. The grouting operator would be misled by the appearance of bleed water on the surface of the grout in the reservoir. This amount of bleed was somewhat consistent with that recorded in the laboratory.

Development of a stable grout

The lack of correlation between bleed tests carried out on small samples and the behaviour of the grout in a long, near-vertical duct, suggested that any grout with a tendency to bleed would form a void at the top of the duct. In addition to developing a technique such that a void could be regrouted when it had formed, a search was undertaken to find a more stable grout.

Many commercial additives were tried and those that showed promise of producing a stable grout under laboratory conditions were subjected to trials on long ducts. Expansive and non-expansive additives, air-entraining and water-reducing agents, flyash and superfine cement were among those tried. Water/cement ratio was varied in the range of 0.38 to 0.43 to maintain a satisfactory flow rate through a standard flow core. All the above additives tried gave a bleed between 1 per cent and 6 per cent in laboratory tests. Simultaneously, through the Cement and Concrete Association and through discussions with the Cementation Company in Sydney and London, it was learned that a cellulose additive, *Methocel*, had been recently developed and successfully used on jobs for the Tasmanian Hydro Electric Commission. *Methocel*, marketed by the Dow Chemical Company, is a synthetic methylcellulose polymer produced in five product types and many product viscosity combinations. It was found that all mixes incorporating *Methocel* gave zero bleed in 300 mm laboratory tests. In 25 mm and 75 mm diameter plastic ducts, 30 m long, containing strand, no bleed and no settlement took place. An extensive testing programme was undertaken to find the most effective type and the best way of using and controlling it. *Methocel* is supplied in powder form. It has first to be dissolved in water prior to the cement being added.

Two mixes were developed, one for normal grouting operations and one for trickling in

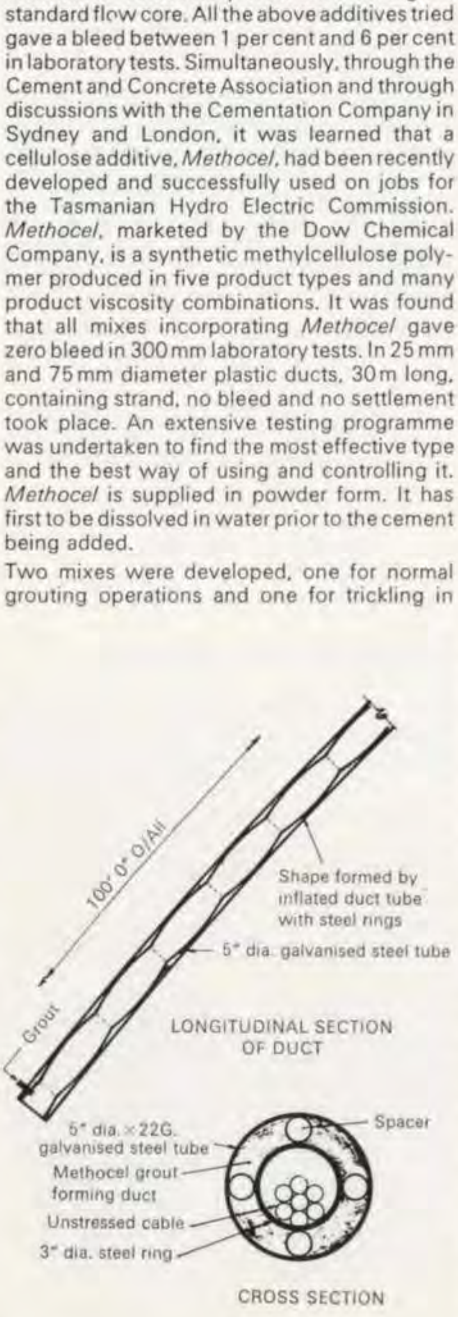


Fig. 2
Details of a model concrete duct

operations where a grout of higher fluidity had to be forced through small holes into the ducts. The most suitable formulation was as follows:

Normal grout:

Water/cement ratio 0.49

Additive: *Methocel* 90 HG 15000 cps DG, 0.39 per cent *Methocel* by weight of water

Mixing procedure: *Methocel* and water are mixed cold in a high-speed mixer for one minute prior to the addition of the cement, then for a further two minutes, followed by low-speed agitation.

Trickling-in grout:

Water/cement ratio 0.50

Additive: *Methocel* 90 HG 15000 cps DG, 0.35 per cent *Methocel* by weight of water

Mixing procedure: As for normal grout.

Investigations on grouted ducts

Various methods were used to investigate grouted ducts for the presence of voids and to determine the frequency and size of such voids. Previously accepted techniques were tried where possible and new techniques were developed and examined. The methods tried were:

1 Drilling
2 X-ray radiography
3 Pulse count using gamma rays
4 Injection of radioactive fluid.

Drilling involves the drilling of a 13 mm diameter hole through the concrete cover and into the duct. It is an operation that must be carefully carried out otherwise damage to the prestressing wire can occur.

At the Opera House approximately 84 ducts were comprehensively investigated by drilling holes at 50 mm, 300 mm or 1 m centres along their complete lengths of 24 m to 46 m.

X-ray radiography

In X-ray radiography an X-ray photograph is taken through the concrete member containing the duct. The source is on one side and the film plate on the other. The time of exposure varies according to the intensity of the source and the thickness of the concrete. It was not suitable for widespread use at the Opera House because many of the concrete sections were too thick; many ducts were in line so that one duct would mask the others and opposite concrete surfaces were not parallel.

Pulse count

In conjunction with Professor H. Greene of the Physics Department, University of New South Wales, what is thought to be a novel technique was developed. A device was built to measure the transmission through the concrete from a radio-active source 137 Cs to a scintillation counter. The rate of transmission was measured as a pulse count on the scintillator. A void would give an increased count. The chief advantages are that it is quick to locate large voids and it was particularly useful in the Opera House investigation in locating the head of the grout. Reinforcement complicates the readings and makes the results somewhat difficult to interpret. Also it is essential to be able to maintain the alignment of the source and counter as both are moved about so that a bracket-holding source and counter must be used.

Radio-active fluid

A fourth method was tried to overcome the disadvantage inherent in the other three methods. During normal grouting operations, cement particles are too large to enter the spaces between the individual wires of a 15 mm diameter strand. The interstices of the strand remain free. This space is small and the area is of the order of 6 mm^2. It was thought possible to pump a radio-active fluid up this central hollow core and to fill any cavities that might exist in the grout by radial flow out between the wires. Thus, a radio-active examination of the surface of the concrete above the ducts with a counter would disclose, by a higher reading than normal, the presence of concentrations of radio-active fluid in the duct.

In this way, voids could be located without having to obtain access to both sides of the duct.

After the technique had been established, the method was not developed further nor used to any great extent on the void investigations because other methods could be used more readily. It does seem to be a valuable tool when access to one side of a duct only is possible and when the ducts are some distance below the surface and not positioned accurately so that drilling is reliable.

Chemical grouting

In the early stages of the investigation it was feared that small voids could occur randomly distributed along the length of many of the ducts. It was clearly impossible to determine where they all occurred by any non-destructive means. The magnitude of the problem was not known but it was thought possible that their occurrence could constitute a hazard.

Working on a tight programme for the completion of the job, a solution had to be found before the full extent of the situation was known. To this end experimentation on grouting these voids was undertaken and carried through to some finality before it was established that the occurrence of intermediate voids did not warrant special measures. Although it was never used in practice it is worth reporting the methods developed and the materials selected in the hope that it may be useful on other problems.

The principle adopted was to pump a fluid chemical grout up the interstices of the wire from one end. This chemical grout should be of such viscosity that it could flow radially through the strand and fill voids anywhere along the length of the strand.

Two things had to be established:

(a) A suitable chemical grout

(b) A technique for injecting it into strands.

Chemical grouts

Many materials or groups of materials were investigated – polymers, hydro-carbons, vinyls, polysulphides, epoxies, etc. The three that showed the most potential were:

1 Epoxy resin – formulated by Shell Chemicals

2 Polythixon – formulated by the Cementation Company

3 Polyester/Monomer – formulated by Monsanto Chemical Company.

Injection experiments were not completed on these three materials as the need for using them had disappeared. However, a considerable amount of field testing had been carried out on injection of water and chemical grout into some model set-ups and into some ducts in the structure. The tentative conclusions that can be drawn from these tests are as follows:

1 To achieve best input, connection should be made to the end of the strand by glueing an intake nozzle on with epoxy. Drilling into the side of the strand proved unreliable for injection.

2 To get reasonable quantities of water-flow along the strand, pressures of 1.4–2.1 N/mm^2 had to be used. For grout with a viscosity of 30 cps a pressure of 2.8 N/mm^2 was envisaged.

3 Grout would not flow radially from the strand into all of the voids and some voids would remain ungrouted as a result. This was particularly the case with smaller voids.

4 It was believed that when the chemical grout entered the void it would provide adequate protection to the cable.

Summary of void formation

Examining the results of the investigation of grouting of the ducts, the following conclusions on the incidence of voids on this structure could be drawn:

1 In the 75 mm ducts containing the seven 15 mm strand, only a top void was formed. No intermediate voids were found to occur below the head of the grout. This was thought to be due to:

(i) Lower pumping speeds – the head of the grout moves up the duct more slowly and there is less tendency to entrap intermediate air voids.

(ii) The bleed water is able to rise to the top of the grout column in a duct of this diameter and not get stopped at intermediate restrictions.

(iii) Where undetected leakage does occur, the size of the leak is significantly smaller than the size of the space within the duct. As long as the grout can flow out through a small crack in the duct wall, then grout will be able to flow down along the duct to replace it with a consequent lowering of the head of the duct. If larger leakage occurred, this was invariably detected and plugged prior to or during grouting.

(iv) There was less chance in this particular structure of inter-connection of the 75 mm duct with other longitudinal ducts.

2 On the 35 mm ducts some intermediate voids were found, randomly distributed along the length of the strand, most of them not extending through to the tendon. The restrictions at the temporary anchorages, where the available passage for grouting was an annulus 3 mm wide, was thought to influence this. The voids appeared to be caused by the following:

(i) Separation of the cement and water (bleed) with the cement arching above the point of bleed

(ii) Cement arching above a leak point because of the small distance between the strand and the wall of a duct

(iii) Air trapped in the void due to inefficient pumping and being unable to rise because of the higher friction in the smaller duct

(iv) Turbulence caused by fast pumping.

Corrosion problems with voids

When voids occur in ducts and expose the steel, an assessment must be made on the extent of the corrosion hazard. Interchange of water and oxygen necessary to promote corrosion could come from one of four main causes:

(a) Down the interstices of the strand

(b) Through cracks in the concrete cover which had occurred before grouting

(c) Cracks after grouting

(d) Through permeation through the concrete cover.

Flow up the centre of the strand is prevented at the ends by the concrete cover to the anchorage. Even if it could take place, then the corrostion products would soon block up the interstices. Anderson *et al*, in their account of construction of the Forth Road Bridge, report a flow of hydrogen gas up the centre of the strand forming the ground anchor to the main cable. This subsequently ceased because of progressive blocking. Cracks occurring before grouting would almost certainly be blocked by the grout itself being forced into the cracks or leaks by the high grouting pressures. This, after all, is an accepted technique in blocking cracks. Cracks formed after grouting would extend through to the strand itself and, if sufficiently large for water and air penetration, would provide a similar degree of hazard whether the void were present or not. Permeation through concrete of the density normally used in prestressed concrete is slow and is a negligible hazard. It is the author's opinion that such small voids as are found intermediate along the length of the cable do not present a corrosion hazard.

Experience on site seems to justify this. The appearance of the strand in those holes which exposed a void reaching to the tendon was similar to that of newly-cut strand. After the drill-hole had been left open in the weather for several days, rusting commenced.

Modified grouting procedure

The modified grouting procedure revolved around the knowledge that leakage could cause a top void in the duct without it becoming apparent to the operator. It was determined then always to form a top void which would subsequently have to be topped up. Ducts were grouted in a two-part operation.

Two lateral holes 300 mm apart were cast into the duct about 1.5 m to 1.8 m from the top anchorage. Using a *Methocel* grout the duct was grouted from the lower anchorage until a grout flow came from the first hole. After 24 hours, grouting from the bleed-hole immediately above the head of the grout took place until it flowed from the grout hole in the top anchorage. This procedure was found to be fully effective.

Filling of the top voids in the already-grouted cables was based on a modification to the procedure.

From a hole drilled laterally into the void, a fluid grout was trickled into the duct to bring the head up to the level of the hole. This grout was first allowed to set and then pressure grouting was used to fill the void above the hole. A pressure of 1.4 N/mm^2 was applied for three to five minutes so as to displace trapped air and the drill-hole sealed off under pressure. In this way a satisfactory column of grout was achieved.

Summary

Apart from the extensive range of testing and investigation which in many cases is not complete, but which could be of value to future investigation, a number of problems have been isolated and a number of satisfactory solutions have been developed. From our experiences in the case, it is possible to draw some general conclusion which could apply to other work:

1 Bleed is aggravated in long vertical columns of grout.

2 If small ducts are used of diameter less than about 75 mm, care must be taken to ensure that separation of the grout column below the head of the grout does not take place.

3 A stable grout showing no tendency to bleed is recommended. The use of a grout containing *Methocel* as an additive was found to be very satisfactory.

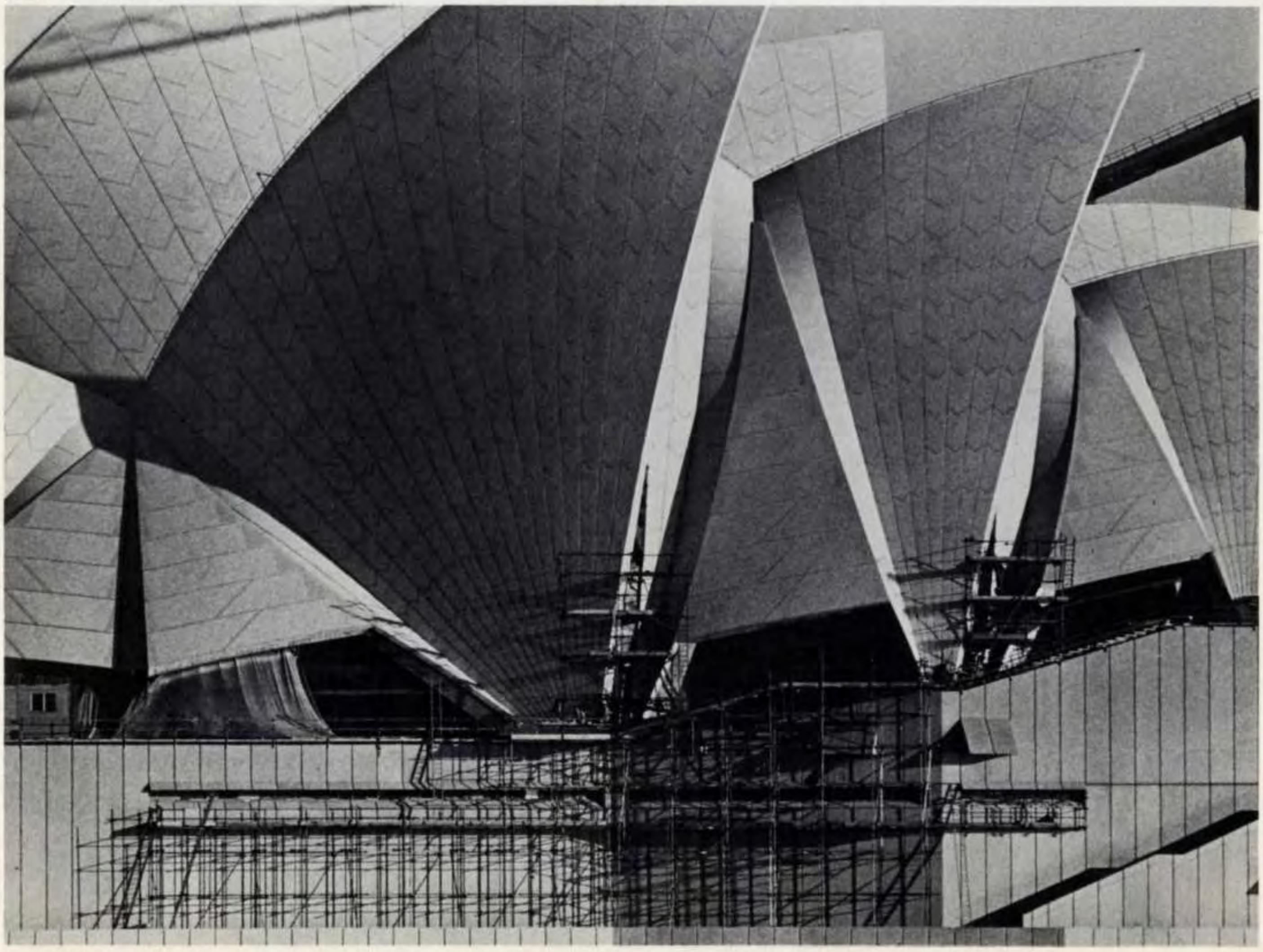
4 Because of the higher-than-normal pressures involved, leakage can occur from zones which would normally be considered sealed. In both lined and unlined ducts, interconnection between the duct can take place at the ends of the precast segments. In unlined ducts, which are frequently used because of their cost advantage, leakage through microcracks or around adjacent fixings or into transverse holes can occur fairly readily.

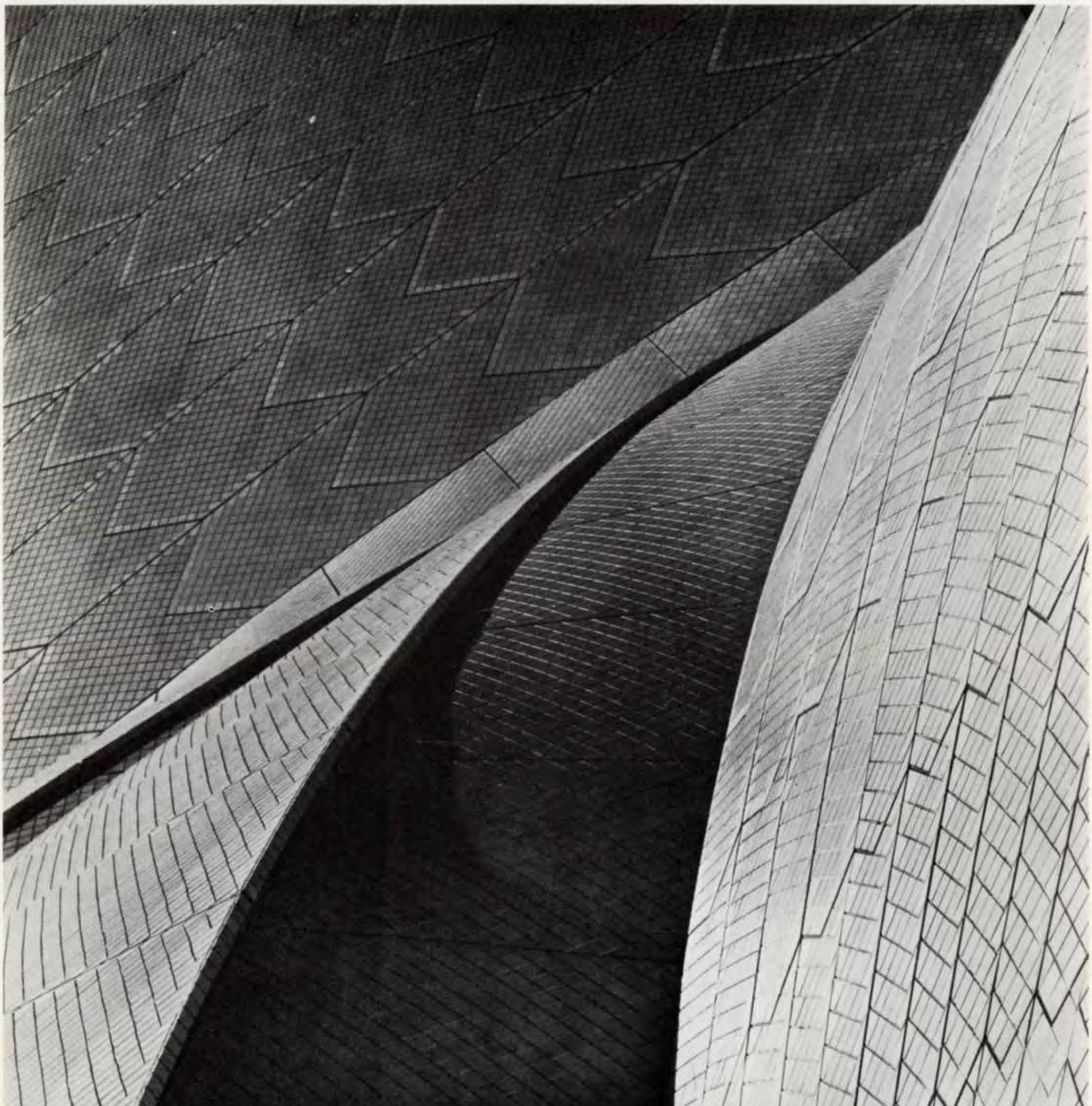
5 Bleed holes where possible should not be of too small a diameter – 19 mm is suggested as a minimum.

6 Drilling into a prestressing duct has been found to be a satisfactory and reliable method of investigation
and finally,

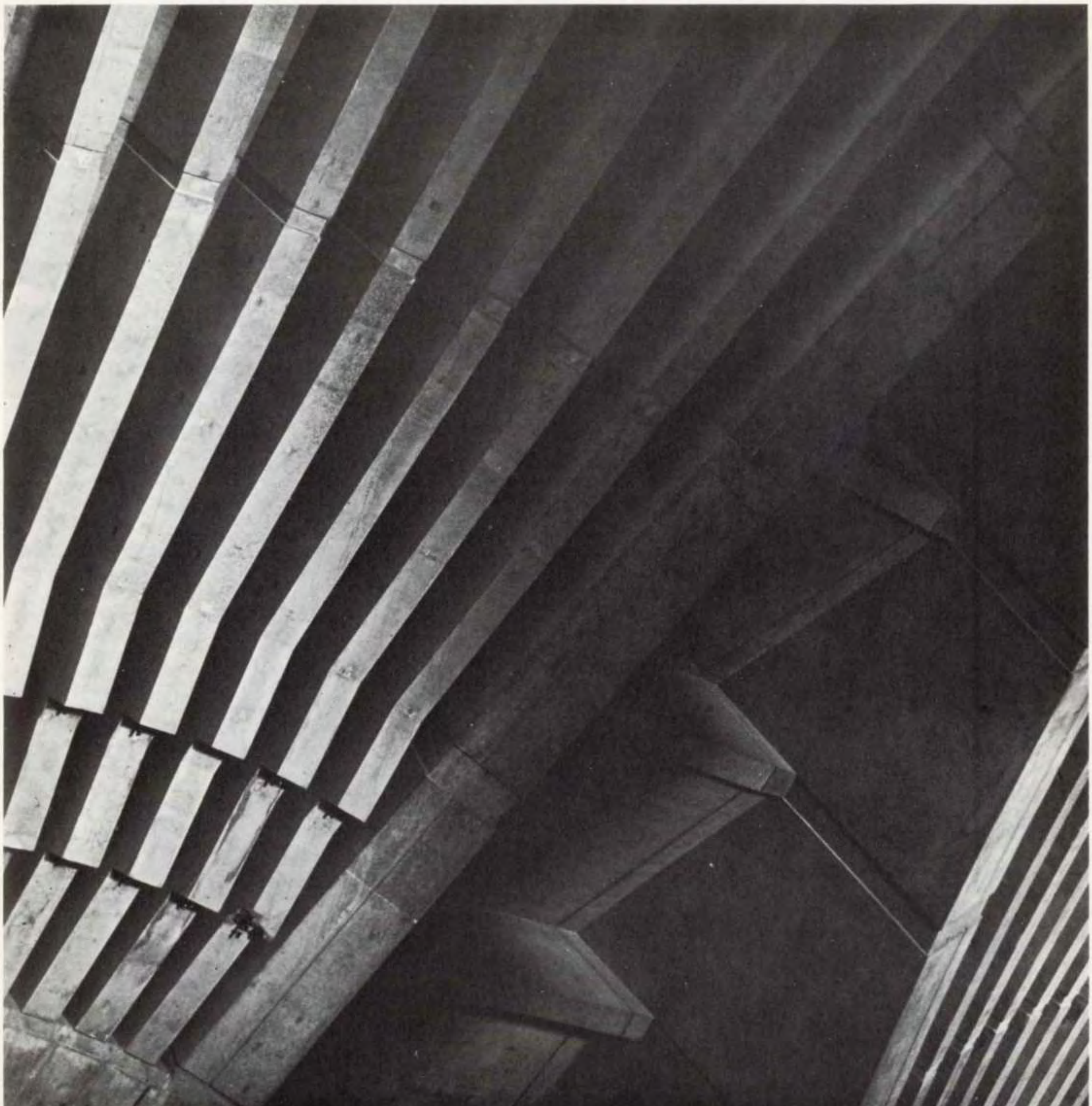
7 The pulse count technique and the radioactive fluid method are interesting methods of investigation which could be of use in certain circumstances.

(Photos: Harry Sowden)





 (Photos: Harry Sowden)



Adhesives for structural jointing

Turlogh O'Brien and John Nutt

Introduction

In the early 1960s considerable interest was aroused in the use of resin adhesives for jointing precast concrete. Prior to use on the Opera House, epoxy adhesives had been used in Coventry Cathedral, and on a bridge in France. However, for any new jointing compound to be used widely with confidence, a detailed knowledge of its mechanical behaviour was required. For resin adhesive materials five categories of behaviour may be distinguished:

1 The mechanical properties of the adhesive material under short-term testing
2 The bond strength to particular materials under short-term testing
3 Fire resistance data of various typical joint details
4 Long-term deflections under various states of stress
5 Long-term strengths of the adhesive in various environments.

Tests were carried out which showed the very good short-term properties that could be obtained. However, absence of data on long-term performance has restricted the structural use of resin adhesives to thin compression joints where the principal function is that of an efficient gap-filler and stress-distributor. Five types of joint have been formed on various buildings:

(a) Thin joints between precast concrete units, which are post-tensioned after the adhesive has cured
(b) Thin horizontal compression joints between sections of precast concrete mullions or columns with no dowel connector bar
(c) Thin horizontal compression joints between precast concrete column sections, with dowel connector bars grouted in with the same adhesive
(d) Precast concrete shear connections using bolts to maintain the adhesive in compression
(e) Various joints used in assembling precast concrete staircases.

In addition to the stress-distributing function, other advantages may be obtained from resin adhesives:

(i) The joints appear as a thin line instead of the 13 mm band obtained with mortar joints.
(ii) Under favourable weather conditions the rate of hardening of the adhesive can assist early de-propping, post-tensioning, etc.
(iii) Shrinkage in the joint is negligible, leading to greater watertightness.

Description of the shell roof

The shells of the Sydney Opera House are formed by curved hollow concrete ribs which lie side by side so that a continuous spherical surface is formed. The ribs are made of precast segments, cast with matching faces, and made longitudinally continuous by post-tensioning across transverse epoxy joints. The segments weigh about 10 tonnes and vary in width from 300 mm to 3.7 m and in depth from 1.2 m to 2.1 m. The epoxy joints are a nominal 1.6 mm thick – in practice this varies from 0.4 mm to 2.4 mm – and a prestress is applied during erection while the resin is still fluid, within 15–20 minutes of application. This prestress creates a uniform compression of 1.4 N/mm^2 over the cross-section.

As the ribs are inclined at various elevations, the angle of the jointing face to the vertical ranges from 15° to 85°. During the erection the precast segments are supported on a travelling steel arch, which is removed as each rib becomes self-supporting. Subsequently the stresses across the glue line are complex and are caused by the applied prestress, the moments, shears and tensions.

The maximum and minimum compression likely under extreme conditions across the joint are 17.2 N/mm^2 and -2.1 N/mm^2 respectively. The maximum shear stress is of the order of 3.4 N/mm^2. Under normal conditions the compressive stress is 10.3 N/mm^2 and the shear stress is 0.7 N/mm^2. The segments were cast, stored and erected in the open with no weather protection.

The thin epoxy joints enabled:

(a) The geometry of the curve of the rib to be accurately set out in the casting yard and duplicated during erection in the air
(b) The ribs to be erected quickly. It is estimated that the time required to make a joint and leave it to cure before full prestressing was about one-fifth of what would be required with cement mortar joints.

The epoxy resin system

The resin used was a polyamide-cured epoxy resin supplied by CIBA Company Pty Ltd of Sydney. Two grades of hardener were used, 63/27W for winter and 63/27S for summer. The former contains an amine adduct as accelerator, the latter has an aminophenol accelerator. The resin, *Araldite* 63/27, contains *Araldite My* 752 (a liquid epoxy resin containing a reactive diluent), 200 mesh quartz flour, *Aerosil* and grey colouring paste. The properties of the two systems are given in Table 1.

Originally an amine cured system was offered at the tender stage. It met the specifications for strength and had a higher heat distortion temperature. However, the better application properties (easier to handle, less sensitive to dampness) of a polyamide were considered to be more advantageous.

Surface preparation

The concrete surfaces to be glued had been cast against each other to achieve matching faces. A bondbreak material was used to prevent adhesion during casting, and this had to be removed before glueing. To check the efficiency of various bondbreak compounds, and to assess their effect on the bond strength of the epoxy resin, a test programme was commissioned from the School of Civil Engineering, University of New South Wales, Sydney. The method used was to glue end-matched half beams to make 0.7 m long beams for testing in flexure. After casting, various surface preparation treatments were tried. These included:

(a) Acid etching with hydrochloric acid, followed by water washing
(b) A detergent scrub and water washing
(c) Wire-brushing and water washing
(d) Grinding away at 0.8 mm layer.

All the commercial bondbreaks and surface treatments were satisfactory in the laboratory, showing little reduction in strength. Field trials were carried out on full-size units using method (a) above. These failed badly, with bond failure over large areas. A stronger acid concentration seemed necessary.

The final procedure adopted, which proved most satisfactory on site using a hydrocarbon resin as bondbreak, was as follows: grind off 0.8 mm of concrete not more than ten days prior to jointing. If this time is exceeded, regrinding is required. Wash down with absolute alcohol immediately before jointing to remove dampness and grease. No resin is applied to damp surfaces, flame drying being used if necessary.

Application techniques

Early laboratory trials showed little difference in strength between priming or coating both surfaces or only one. However, later tests showed that application to both surfaces was marginally preferable. The quantity of resin required for each joint was calculated and a small percentage extra allowed for squeeze out. The epoxy was applied by trowel to the lower surface, spread so that there was some additional thickness at the corners and worked around the spigots and ducts. (It was prevented from entering duct and bolt holes by means of a cork gasket glued to the lower surface). A roller was used to apply a thin layer of the same epoxy to the upper surface.

Adjacent surfaces were protected by a polythene skirt fixed by masking tape which extended 6 mm onto the jointing surface. All segments were treated with a light coating of a PVA emulsion prior to erection to prevent adhesion of spillage from above. This emulsion weathered off in time.

Shimmed joints

With matching faces it is sometimes necessary to accommodate angular changes at the joint to maintain correct alignment. This may be done by forming a tapered joint using the following technique:

(a) Steel shimms are used to control the direction of the rotation (the area and position of the shimms would be such as to prevent spalling or local crushing of concrete).
(b) 3 mm steel spacers are glued to the surface whenever an epoxy thickness of more than 3 mm occurs. The shimms and spacers should cover about 60 per cent of the area and the effective epoxy thickness would not exceed 3 mm.
(c) The calculated amount of epoxy required would have an allowance of 50 per cent for squeeze out.
(d) The application procedure would be otherwise similar to that of standard joints. This technique was used whenever necessary. The restricting of the glue-line thickness was considered necessary to limit the total amount of creep.

Scabbled joints

One set of joints were inclined at 70° to the axis of the ribs, and a shear stress due to the prestressing equal to 0.35 of the longitudinal stress existed. Owing to the danger of cracking of the epoxy joint during erection, and because the epoxy/concrete coefficient of friction in attached joints is about 0.35, the joint was lightly scabbled. The surface preparation was to clean the scabbled surface with a wire brush to remove any loose pieces of aggregate, dust off with a soft brush and clean with alcohol in the usual way.

About 0.1 m^2 of unscabbled area was left at each joint and across this a prestress of only 0.2 N/mm^2 during jointing was applied. On some of these joints insufficient resin was trowelled on and some downward flow occurred. After stressing, a gap was left in the joint. The repair technique was to create shear keys in the joint by drilling six 25 mm holes at 50 mm centres, filling with epoxy and forcing 25 mm steel rods into the holes, thus squeezing the fluid epoxy into the surrounding joint. Grouting with a special epoxy was also carried out to completely fill the joint. The shear keys were used as the bond strengths of epoxy to fully cured epoxy is not good.

Other erection problems

At an early stage in the job, trouble was experienced with the jointing of some segments of one of the main arches. Insufficient prestress was applied across the fluid epoxy joint to hold the precast segment in position and angular rotation took place at the joint. This created internal voids within the epoxy layer covering up to 50 per cent of the total area. This was not

Fig. 1
Grinding the surface of a precast concrete rib segment prior to erection, and placing the masking polythene and tape
(Photo: Max Dupain)

Fig. 2
Lifting a rib segment into position
(Photo: Max Dupain)

Fig. 3
View down a rib under construction (Photo: Geoffrey Wood)

Fig. 4
Holding a rib segment in place while glueing is carried out (Photo: Max Dupain)

Fig. 5
Applying the epoxy resin adhesive to both surfaces (Photo: Max Dupain)

noticeable from the outside. However, demounting revealed large unbonded areas of resin/concrete interface.

In addition, movements of the scaffold lead to cracking of some joints. These were repaired by an injection technique:

(a) Drill holes into joint at 300 mm centres

(b) Seal the full perimeter length with epoxy resin putty

(c) Protect surrounding concrete against leaks and bursts

(d) Inject epoxy resin grout from the bottom.

Control testing on site

In addition to their use for rib segment jointing, epoxy resin formulations have been used for patching mortars, ceramic tile adhesive, sealant primer, drill hole fillers, waterproof membranes and joints. It was therefore necessary to have careful control to ensure that the correct mix was used for each job.

This control was achieved by carrying out exotherm and density tests on each batch of material. The measurements of peak exotherm temperature and time were carried out in accordance with the Society of Plastics Industries Specification ERF 2–61 method C. The temperature for each test was 21 °±1 °C. The density tests were carried out using BS 2782 Part 5: 1965, Method 509A. The samples were demoulded four hours after they had gelled, and were cured for 48 hours at 21 °±1 °C.

Limits were specified for each resin formulation giving the range in which the values must fall. In the case of the exotherm test the maximum time and minimum temperature were the important limits. The acceptance limit for density was ±5 per cent of the nominal value. If either test failed, flexure and Shore hardness tests were carried out.

It is believed that these controls enable the following errors to be detected: labelling error, wrong components, omission of components (e.g. accelerators) and the presence of moisture in fillers.

General comments on resin adhesives

The experience of the use of resin adhesives has led to the conclusion that they have very definite advantages for jointing precast concrete units.

However, certain comments must be made about the use characteristics of the materials.

1 Bad joints can look good and so they are difficult to detect. Unless the joint can be tested for strength it is essential to set up a procedure technique and site supervision such that bad joints are unlikely to occur. There is a need for non-destructive testing methods that can be applied in the field. There is no fail safe with epoxies in general. Either bond is achieved or it is not. Because poor joints may look satisfactory, a degree of complacency can develop which makes detection difficult.

2 It is difficult to keep surfaces clean when mechanical lifting equipment is used. Handling of the surface can occur unnoticed. The effect of a small amount of grease from handling is not known.

3 The degree of care required for successful use of epoxy formulations is higher than that of standard building practices. Specially trained personnel are required for the wide use of resins on construction sites (for jointing, patching, sealing, etc). There is considerable danger that malpractices will be undetected.

4 Rapid changes of ambient temperature may cause difficulty in selecting the right grades of adhesive.

5 Little is known of the effect of weathering of a prepared concrete surface on the adhesion of a resin. At Sydney the evidence suggested that a limit of ten days should be put on the interval between grinding and glueing. This appeared to work satisfactorily.

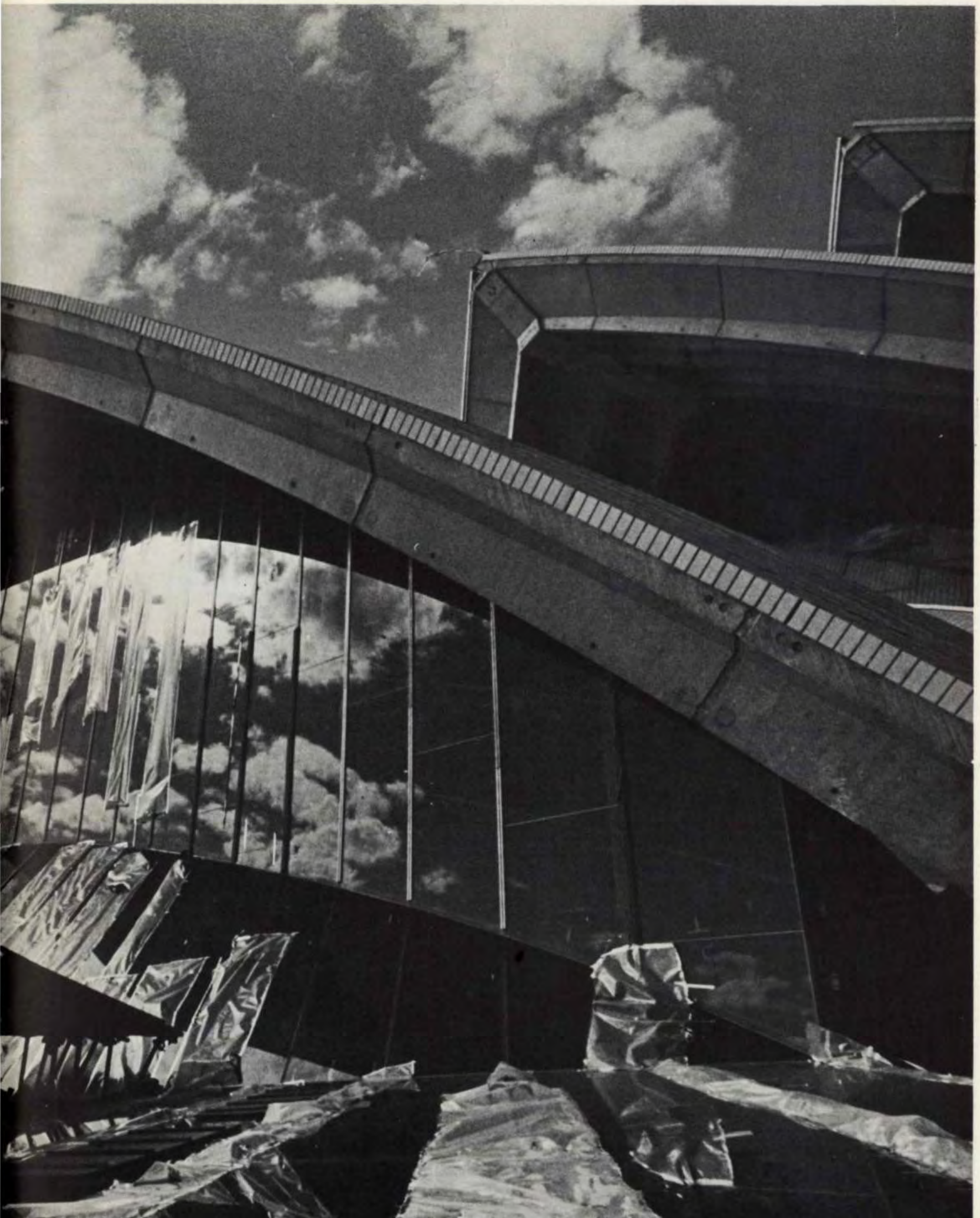
6 Account must be taken of the fact that joints made between precast concrete units with resin adhesives are more rigid than cement mortar joints. Attempts to readjust alignments after the resin has gelled can lead to cracking of units, instead of minor cracking of mortar.

It is quite clear that considerable research needs still to be done on the various topics outlined in the introduction. Only with more detailed information will designers consider the wider use of structural adhesives.

Table 1

	63/27W	63/27S
Mixing proportions by wt	100/22	100/17
Compressive strength, 16 hours at 25 °C (N/mm²)	50.3	41
Compressive strength, 7 days at 25 °C (N/mm²)	72.4	73.1
Tensile shear strength, 16 hours at 25 °C (N/mm²)	6.5	4.7
Tensile shear strength, 7 days at 25 °C (N/mm²)	12.8	12.8
Pot life of a two-pint kit:		
5 °C	2 hrs	3 hrs
25 °C	45 mins	1½ hrs
32 °C	15 mins	30 mins
Heat distortion point	71 °C	68 °C
Reduction in compressive strength after 2 hrs at 250 °C	4%	4.5%
Viscosity of mix at 25 °C (Brookfield no 4 at 20 rpm)	19,000 ±1500 cps	30,000 ±1500 cps

(Photo: Harry Sowden)



Influence of corrosion on some aspects of design of the Sydney Opera House

John Nutt

This paper was presented at the Seventh Annual Conference of the Australasian Corrosion Association, held at Manly, New South Wales, on 7–11 November 1966.

The influence of corrosion extends to many facets of the design of a building, particularly a building of the character of the Sydney Opera House. Its effects have to be taken into account in every project by engineers, both directly and indirectly. It decides, for example, how much concrete cover is given to reinforcement; on grouting procedures for filling ducts containing stressed-steel cables; on the selection of the cables themselves; on what metals can be cast into the concrete, and how two dissimilar metals must be separated when in close proximity to each other. Most of these effects are dealt with by routine procedures, and integrated into the design many times in the course of a scheme. But occasionally special problems occur which, by their very nature, set themselves apart from the normal, and special investigation must be undertaken to obtain a solution.

It is an indication of the nature and complexity of modern building operations that the structural engineer must turn, not only to the corrosion specialist and the metallurgist – as in this case – but also to such people as polymer chemists, to atomic physicists, to adhesion technologists, to meteorologists, to acoustical engineers, as well as to a host of people whose association is more traditional.

Rather than comment on the whole range of normal building processes involving corrosion, this paper will deal in some depth with two such special problems.

In order to put these problems in their proper context, the way the Opera House roof has been designed to act should be explained in some length.

The roof of the Sydney Opera House is really three separate structures over various halls and facilities. Each is formed by the complex interaction of numerous surfaces, all of which are based on a sphere of 75 m radius.

A description of the Major Hall roof will be sufficient to explain the principles involved. There are three major structural units comprising the Major Hall roof (Fig. 20a, p.10), each formed by a pair of main shells and by a side shell which encloses the space between the two main shells. The junction between the side and main shells is formed by large arches which are braced by the beams of the side shell so that the side shell sub-structure becomes a stiff pyramid. The main shells are supported in pairs on this pyramid.

Imagine a thin slice of a sphere, like that formed by two lines of longitude on a globe. The spherical shape of the main shells is formed by ribs defined by just such lines, all meeting at a 'pole' near the foot of the shell. The components of each concrete rib are made in a pre-casting yard and divided into segments which are erected one at a time. These segments are 4.57 m long, weigh about 8 tonnes and vary in width from 300 mm to 3.66 m. Because the ribs are formed by identical slices, all the ribs can be cast in one long mould to different lengths; that they were not was because of the demands of time and not of geometry. The segments are joined together by prestressing along the length of a rib – a technique by which steel tendons carried in ducts in the ribs are tensioned and anchored, thus creating a compression across the joints of the segments. Erection takes place in a shell one rib at a time. During erection the precast segments are carried on a travelling steel erection arch which is removed as each rib becomes self-supporting. The inclination of the ribs varies so that in some positions it lies back along the already partly-completed shell; in other positions it tries to fall away.

Bolting of the ribs laterally at specified points is needed. This bolting system is described later.

Each half of a rib is located at the top by a portion of the ridge beam. The whole structure is covered by precast chevron-shaped panels, containing ceramic tiles and known as tile lids. The forces acting on the structure have been comprehensively analysed. The following loading conditions have been catered for:

Self-weight

Tiles and tile panels

Wind load

Building settlement

Temperature variations (daily, between seasons and over long periods), including the effects of temperature gradients

Creep of concrete

Shrinkage of concrete

Construction loading

Construction sequence.

Table 1

Mechanical properties and corrosion characteristics of some copper alloys suitable for structural applications

Description	Nominal composition				Typical mechanical properties 0.1% proof stress N/mm²		Tensile strength N/mm²		Elongation %		Corrosion resistance in atmospheric conditions[1]	
	Cu	Zn	Sn	Others	Soft	Hard	Soft	Hard	Soft	Hard	General corrosion	Stress corrosion
Copper	99.9				45	95	215	295	50	10	Good	Susceptible to ammonia environments, e.g. bursting of copper heating pipes in floor screeds fixed with ammonia-based adhesives
Copper zinc alloys												
Cartridge brass	70	30			75	370	325	540	55	5	Good. Under some conditions of sea water attack, dezincification can occur	Susceptible to ammonia environments coupled with high humidity and temperatures, i.e. industrial atmospheres
Naval brass	62	Bal.	1.2		95	185	370	400	50	25		
Manganese bronze	57	Bal.	1	Mn 1 Fe 1 Al 0.5	60	280	430	620	30	20		
Copper tin alloys												
Phosphor bronze	92		5	P trace	125	540	310	650	50	10	Excellent	Good
Gunmetal	Bal.	2	10		125		265–340		15–20			
Copper nickel alloys												
Monel	29			Fe 1.25 Ni 68.5 Mn 1.25	230	620	465	695	30	15	Excellent	Excellent
Copper aluminium alloys												
Aluminium bronze	Bal.			Al 10 Fe 3 Ni 1 Mn 1	170–215		495–570		20–40		Excellent	Good

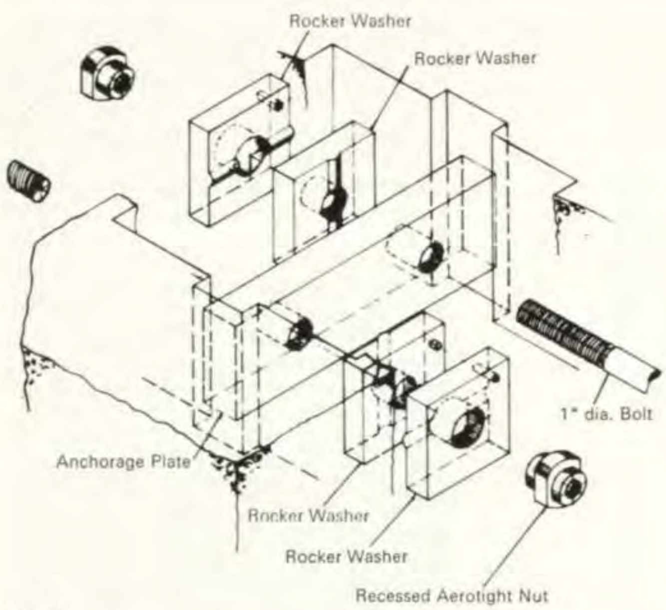


Fig. 1
Typical bolt assembly

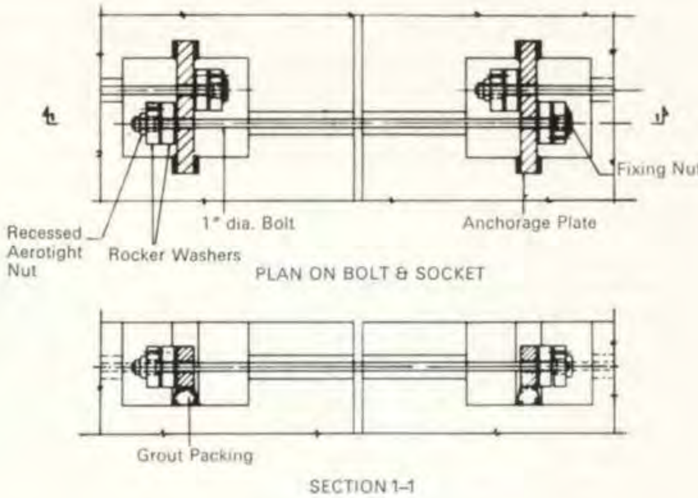


Fig. 2
Bolt assembly plan and section

It is worth noting, for example, some aspects of the forces set up during the construction sequence. This necessitated a complete investigation into the state of stress of the structure at each stage as some units were added, false-work was removed and re-erected, and so on.

Since all 140 ribs are different when erected each one has to be considered afresh.

The sequence of construction imposed one set of conditions upon the lateral connections, viz., each rib should move independently of the adjacent rib. The connections were therefore designed so that in the first instance, ribs are joined to each other by sliding bearings, transmitting in some cases tensile, in others compressive forces, but never shear forces. Rigidity between ribs is obtained later by packing the joints between ribs with concrete and prestressing laterally. The two systems exist together in the final structure, the bolting-sliding pad arrangement acting as a 'fail safe' mechanism to the lateral prestressing system.

The loads in the bolts are quite large and the space in which they can be installed is limited, so that the size of the bolts plays a critical part in the positional tolerance available for their installation. It was therefore necessary to keep the size of the bolts as small as possible. A maximum design figure of 430 N/mm² was used. The factors affecting the design can be summarized as follows:

Fatigue

The amplitude of stress variation in a bolt is 120 N/mm² occurring at a maximum frequency of once a day, though a fluctuating stress of twice the calculated value might occur say 10 times a year.

Corrosion

The conditions under which the bolts would operate are: atmospheric conditions, including the effects of salt deposition and the presence of SO_2 and NH_3 as found in the atmosphere of industrial cities; alkaline conditions resulting from water in contact with concrete and grout. The material had to show good resistance to:

(a) Atmospheric corrosion
(b) Chemical corrosion
(c) Stress corrosion
(d) Crevice corrosion
(e) Cathodic corrosion.

Fire:

In the event of a fire the building is required to stand for a minimum of two hours. Protection is offered by the surrounding concrete and a 50 mm asbestos layer placed around the assembly. An analysis showed that the temperature could reach 400°C in this time and the material chosen should be able to maintain adequate strength at this temperature.

The materials considered for the purpose were:

1 Stainless steel *FV520B:* 13% Cr, 6% Ni, a martensitic stainless steel, which gains its strength from precipitation hardening
2 *Monel K-500:* 70% Ni, 30% Cu gaining its strength from cold working and subsequent heat treatment
3 Titanium *314A:* A 4% Al, 4% Mn alloy with a moderate amount of heat treatment
4 *Nimonic 90:* A high-strength nickel–chromium–cobalt alloy.

The resistance of the various materials to the different forms of corrosion can be summarized as follows:

Atmospheric corrosion: Titanium has the best resistance. Stainless steels are liable to pit and stain and, with the exception of *FV520B*, were rejected.

Stress corrosion: For the conditions that exist this was not thought to be a serious consideration although stainless steels are subject to stress corrosion in the presence of chlorides.

Crevice corrosion: Monel K is liable to suffer to some extent from crevice corrosion but it was thought that it might be a more serious drawback for stainless steels. *Nimonic 90* must be considered the worst because of the difficulty of working the material. Titanium does not suffer from this form of attack.

Cathodic corrosion: This could be avoided by good detailing of the components.

On balance the order of preference on the corrosion properties appeared to be:

1 Titanium *314A*
2 *Monel K*
3 Stainless steel *FV520B*.

Monel K has superior heat resistance properties and is cheaper. Titanium was thought to have superior fatigue-resistance properties.

Typical mechanical properties of titanium and monel are:

Titanium *314A:* 0.2% proof stress – 960 N/mm²; ultimate tensile strength – 1050 N/mm²; elongation – 19%

Monel K-500: 0.2% proof stress – 865 N/mm²; ultimate tensile strength – 1205 N/mm²; elongation – 19%.

A programme of testing was devised to check the fatigue and corrosion resistance of complete assemblies made of titanium and *Monel K*.

Fatigue tests were carried out at Cambridge University in a 60 tonnes capacity machine at a frequency of 2000 cycles per minute. The results showed that for less than 1 m. reversals *Monel K* exhibited superior performance, and above 1 m. reversals titanium was better. Since the specification called for 100,000 reversals (based on a daily variation over 300 years) *Monel K* was preferred.

Accelerated corrosion tests were carried out in Messrs. Sandberg's laboratories in London. Four full-sized assemblies were made up, two in each of *Monel K* and titanium. Each was stressed to 695 N/mm² and positioned in a mist chamber. One of each pair was protected from direct contact with a salt spray by a polythene wrapper.

The salt spray solution contained the following concentrations of salt:

Sodium chloride 2.7%

Magnesium chloride 0.6%

Calcium chloride 0.1%

Potassium chloride 0.1%.

A corrosion cycle consisted of exposure to the salt mist for two hours; drying and maintaining at a temperature of 33–37°C at a relative humidity of 90–95% for seven days. The testing was continued for 238 days. All assemblies were free of corrosion at the end of the tests apart from some tarnishing on the *Monel K* bolts. This was most marked where the bolt had not been protected from the direct effects of the salt spray.

Monel K-500 was finally selected on the grounds of cost and resistance to fatigue. A typical bolt assembly is shown in Fig. 1 and Fig. 2. The rocking washers, two at each end, are designed to give the necessary freedom of movement and to keep the direction of the stress axial to the bolt length. A fixed nut is screwed to one end of the bolt and an *Aerotight* nut is used at the other. The nuts are of special design to reduce high-stress concentrations in the threads of the bolts.

In the design of the tile fixings, the resistance to corrosion influenced the selection of the materials for the components and the stress levels at which they were designed to act.

The tile lids are large units weighing up to 3 tonnes which have to be supported in a range of orientations. It was architecturally necessary to achieve continuity of surface from one tile lid to adjacent lids, and in addition such continuity would ensure that the waterproof sealing of the joints between the lids was carried out effectively. Because the rib structure on which the lids are attached is not erected to such fine tolerances, the fixings have to be capable of absorbing the differences.

Parts of the fixing were required to be cast into the concrete during the casting operations of both rib segments and tile lid. There must be no maintenance required and the effects of temperature expansion and contraction must be catered for.

Details of a typical fixing assembly are shown in Fig. 3.

It became apparent that the selection of materials and suitable stress levels would be determined by considerations of stress corrosion. A number of failures due to stress corrosion has been reported in recent years in the UK on fixings made of manganese bronze, and, as this material has been widely used because of its high strength, it is likely that there will be future failures.

Stress corrosion involves the simultaneous action of static tensile stresses and a corrosive environment. The degree of concentration of the corrosive environment may vary with humidity and temperature and the applied stress may be caused by the internal residual stress locked in during manufacture. The effect of an intermittent combination of the two conditions has not been established, nor has an optimum stress value been determined which will ensure against stress-corrosion cracking. The effects of stress variation on stress corrosion and the interaction of fatigue and stress corrosion are largely unknown.

The information important to the designer is that given a particular location and hence a probable environment, what are the stress levels that can be used? The general concensus of opinion is that the best means of prevention is by selection of a suitable alloy and limiting the maximum tensile stresses to the range 0.25–0.5 of the 0.1% proof stress. Account must be taken of the stress concentrations, possible residual stresses and unsuspected high concentrations of the corrosive environment.

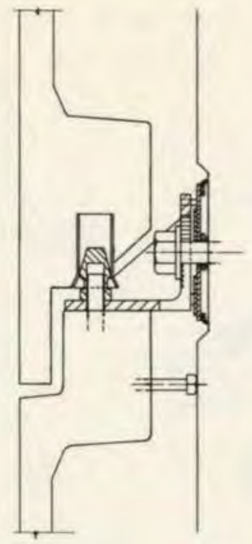


Fig. 3
Typical Fixing assembly

Table 1 is a summary of the stress-corrosion performance of some copper-based alloys.

The Sydney atmosphere varies considerably according to location, but it is probable that with time it will develop to be as polluted as the air of any large industrial city. Thus it would contain proportions of sulphur dioxide, ammonia and chlorides.

It was therefore recommended that the following materials should be used for the components of the fixings:

(a) For rods: phosphor bronze to *BS369* (hard), stressed to 140 N/mm².
(b) For castings: aluminium bronze to *BS1400* –AB2–C, stressed to 90 N/mm².

The two problems described here are unique problems of a type not encountered in day-to-day structural engineering. But they illustrate the reliance the structural engineer must place upon his specialist advisers in order to make decisions in the design. Unfortunately it is not always possible to examine in such depth the problems which occur, nor is it easy to locate the correct specialist advisers; and to fill this knowledge gap, there is a need for the production of concise expert reviews to cover aspects of design which rarely occur. Whether such accounts would ever be found again amongst the explosion of papers produced is difficult to say. But this is another matter not to be dealt with here.

Reference

(1) TOWNSON, V. G. Resumé of information on stress corrosion cracking of metals most commonly used in structural engineering. Unpublished research report, Ove Arup & Partners, London.

Sydney Opera House Awards

The following awards have been made to Ove Arup & Partners:

1969

Queen's Award to Industry (for technological innovation in prestressed concrete roofing).

1972

Association of Consulting Engineers of Australia Annual Award for Excellence (for the design and construction of the glass walls).

1973

Institution of Structural Engineers Special Award to acknowledge a physical achievement in structural engineering in its widest sense (for our contribution to the creation of the Opera House).

Credits

Constructing Authority:
Minister for Public Works
Government of New South Wales.

Architects:
Stages I and II – Jørn Utzon.
Stage III – Hall, Todd & Littlemore.

Consulting engineers:
Ove Arup & Partners.

Main contractors:
Stage I: Civil & Civic Contractors Pty. Ltd.
Stage II: M. R. Hornibrook (N.S.W.) Pty Ltd.
Stage III: M. R. Hornibrook (N.S.W.) Pty. Ltd. (Co-ordinating contractor).

MacDonald, Wagner and Priddle, Consulting Engineers, of Sydney, were associated with Ove Arup & Partners in the supervision of Stage I.

Editor's note

The object of this issue of the *Arup Journal* is to bring together, in one convenient package, six of the most significant papers to have been written about the Sydney Opera House.

Because these papers are reprints of articles from several different periodicals some duplication of subject matter will be found. We have, however, avoided duplication of drawings and photographs by making cross-references between the relevant articles.

(Photo: Harry Sowden)