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SESSION 4

INSPECTION, MAINTENANCE AND OPERATION

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The Computer as an Aid to Better Design

L'ordinateur - auxiliaire en vue d'un meilleur projet

Der Computer als Entwurfshilfe

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SUMMARY

For tension structures, where form is so closely allied to the structures ability to perform, architect and engineer must be able to communicate their individual perspectives to create a total design. Different education and goals makes communication difficult. Computer models which can be developed and used by the whole design team are now providing a common "model" which is the key to improving the communication that is vital to creative design.

RÉSUMÉ

Les structures sollicitées à la traction, dont la forme est étroitement liée à l'efficience, impliquent que l'architecte et l'ingénieur puissent se transmettre leurs propres points de vue, afin de réaliser un bon projet. Mais les différences de formation et d'objectifs d'étude ne facilitent pas ces échanges. A l'heure actuelle, des techniques assistées par ordinateur et utilisables par toute l'équipe de concepteurs permettent de fournir un "modèle" commun, qui représente la clé de communication indispensable pour la créativité de l'étude.

ZUSAMMENFASSUNG

Für zugbeanspruchte Tragwerke, bei denen die Form so eng mit der Leistungsfähigkeit verbunden ist, müssen der Architekt und der Ingenieur ihre individuellen Sichtweisen mitteilen können, um einen geschlossenen Entwurf zu erreichen. Unterschiedliche Erziehung und Entwurfsziele erschweren dies jedoch. Computertechniken, die vom ganzen Designteam benützt werden, können heutzutage ein gemeinsames "Modell" liefern, das den Schlüssel zur Kommunikation darstellt, die für den kreativen Entwurf unabdingbar ist.

At FTL and Buro Happold we have been working for the past fifteen years to develop software which produces a model for the architect, structural and services engineer. At the core of the software is a dynamic relaxation routine which defines a surface form and forms the base for stress analysis of this surface under applied loads. Input of a model requires only the definition of a few key parameters and the geometry of fixed positions along the perimeter. A network of triangular membrane elements is generated automatically and the form (surface geometry) is defined when equilibrium is achieved under a given internal stress pattern - there is a close analogy with a soap film model at this stage. The model surface co-ordinates are stored but rarely need to be known. By attributing "real" values of stiffness to the membrane elements and " real" loads perpendicular to the surface to model wind or snow, new equilibrium state can be defined which give the designers a good measure of the likely deflections and stresses both in the membrane and other key components.

The surface geometry is also vital in providing cutting pattern data for the fabricator, as the skin must be made from a number of tailored panels of membrane to achieve the final curved form once stressed. The fabricator uses the same model as input to his automated cutting table.

The model which serves the engineers and fabricators can now also serve the architect. High resolution screen graphics allows a model to be built up around the defined membrane surface which is an accurate representation of the whole local environment. Rapid refreshing of the image gives the user a "walk-through' option which goes beyond that possible with a physical model. By reading in digitised images, real life and model can be superimposed to give a life like image. At this stage the model can be refined to investigate any combination of alterations to the form of the structure and its surroundings. This has implications for the whole design team, as it is of the utmost importance that they all relate to this central model and are involved with such developments at all stages.

Detail design can again develop out of the core model which is a store of all the geometrical and stress requirements of the structure. Selection of elements from a library of available components allows the rapid visualisation of suitable contributions and to a suitable design choice. Full scale drawings can be output from the model. Component schedules for ordering can be taken off for the fabricator.

Buro Happold has worked with many architects offices in developing tension structure designs for projects all over the world. Recent collaboration with Ron Herron led to the production of an award winning design for an office refurbishment in Central London for the company 'Imagination', including a free-form fabric roof covering a central atrium and exhibition gallery. Heron Associates are in the front line of the development of advanced visualisation techniques for architectural design using relatively simple desk top computers.

The design of the Imagination Building took place in 1988 when the architects and engineers developed their models separately. Communication was by two dimensional images alone.
Since then both practices have developed more advanced software and focused more attention on the easy transfer of compatible data between offices. We are now able to work effectively on a single common model. A surface is developed initially to an equilibrium form by the engineers who can also incorporate visualisation options. This surface model is then transferred to the architect who incorporates it in a more detailed local contextual model with any amount of "paste on" imagery. Meanwhile the same model is developed by the engineers for load analysis and patterning.

Architecture and engineering - a problem of language:

Engineering as we know it today is much younger than architecture. From its start as a profession, engineering was seen as built round a body of knowledge as scientific as possible. This is given to young recruits at University - after all, what are academics for but to add to that central core of scientific knowledge - and then learn how to apply those principles systematically under an apprenticeship to a skilled practitioner.

The architects have taken another view. They have consistently hung on to the concept of education by apprenticeship even within the Universities. The student carrying out project work which is constantly being compared with precedent encourages a broader, less structured approach more related to an education in the humanities, based on scholarship in the critical study of historical buildings.

Thus an architect will describe his aim as "giving one's client what he wants but does not know he wants it" while an engineer can describe his aim as "doing for one buck, what any fool can do for two". An appeal to a broader sensibility by the architect, a pragmatic solution by the engineer.

In this they are reflecting their separate responsibilities. Design is the organisation of building. It is cheaper to think out what you are going to build before you start building it. It denotes both the content of a set of plans to build from and the process of the production of those plans. The process involves tentative layouts of the building and its components, checking of the structure and servicing by numerical analysis and the necessary testing before fully detailed drawings are released for construction. It is a complex iterative process in which the responsibility for the overall drawings lies with the architects, and the responsibility for the performance of the structure and services - together with determining their construction -lies with the building engineer.

Galbraith has written "what is common to most successful technical enterprises is the inevitability of collective decision making and guidance in which specialists participate, contributing the needed knowledge or expertise." That decision making is dependent on dialogue, in turn dependent on enough shared language; understanding not just the concepts but the intentions.

If one thinks about this it is incredibly difficult. Even if the disciplines have the same common everyday language the architects design techniques are primarily visual, the engineers are usually either experimenting physically or analysing numerically. Further to that, there is often a deliberate educational separation promoting differing professional values and mystification. An example of this is that, with the exception of a very few practitioners, those engineers most admired by architects are not the same as those most admired by engineers and vice versa.

The nature of innovation - the development of techniques:

In the complex world of building there is relatively little innovation - and what there is probably develops slowly. But it exists in what we will call "radical design", where either the total solution is new and original or, much more likely, some aspect of the problem is. In any profession there are relatively few practitioners interested enough and able enough to attempt radical design. The building engineering radical usually has to fund it himself. It is rare to be paid to do it. Radical design requires considerable courage since one is "selling" an unproved idea to a client, committing oneself into carrying out an indeterminate amount of original work in the same time or less than would be allowed for a normal design and acceptance of responsibility for construction and performance. If it is a brilliant idea it will soon be used and claimed by others, as design developments in building cannot easily be patented. To be successful at it one has to work at it over a period of time; successful innovation occurs in a series of steps, we are only describing a small one in this paper. The generator is not, we think, primarily commercial but , as Francis Chichester said "it makes life more intense".

If achieving increased efficiency is an engineer's contribution, a radical architect still has to join in it. The risk is probably different as are the skills required and there is still an enormous amount of unpaid work. A new technical solution often leads to a new aesthetic and while change is an important element in architecture (how else will the young architect kill off the old?) there are very strong fashion currents which have a very slow relationship to technological advances. The engineering must bring restraints and a whole range of spatial and planning problems for which there are few precedents. The reward though is that when one looks back at a truly radical building design it will have faults but usually also an immense impact and life to it. It is as though, starting with the strive for economy, a clear pattern or order is produced which is taken down a hierarchy of detail to achieve a sense of organisation and unity.

Such innovation tends to follow a standard pattern. Developments in material science or similar stimulate more understanding. A first design is usually made by physical modelling, with interactive testing being carried out between the small scale and the full scale. Numerical analysis of behaviour will follow but its accuracy only improves as understanding of the scale factors in the physical modelling develops.

All designing is modelling; almost all of it is analogue modelling. It is the "caricature" element of the model which informs the designer. Thus a soap bubble model of a tent shows, nearly, the minimum surface area, with the implications of economy. A tulle model, especially with modelscope photography to remove the scale effect, can express the spatial qualities. And so on.

But such physical modelling is expensive of time and effort, and it is developing numerical modelling which shortens design time, reduces design cost and makes more generally available the knowledge of how to design such types of structure.

Large scale tensile buildings are a relatively recent development providing cheap volume space capable of resisting enormous forces but with limited architectural precedent.

Such tension structures rely on pre-tension in their surface to stabilize them under fluctuating loads and enable the membrane to handle "compression" by a reduction in tension. Generally, the surface is pretensioned against itself, having anti-clastic curvature (saddle forms). The geometry (or form) of this surface determines the way the membrane distributes stresses.

The surface is the structure. There is no distinction between the surface of the building and the skin which carries the loads. The skin defines the architecture. It is this interdependence which brings together the architect and engineer and demands a mutual appreciation of the design opportunities and constraints. The key to this is communication without which the design process fails. It is the techniques developed for communication in the design process which we now wish to examine.

The Core Model: The struggle for an overall technique:

The traditional key to communication in building design is the physical model. In past centuries physical models were often the primary tool for communication. Decisions were made after making prototypes and mock-ups and then translated into drawings. For tension structures physical modelling has long played an important role. Such models bring together the architect and engineer by using materials, such as soap films or woven nylon, which will take up forms that relate directly to the surface stresses. In this way they relate both to the aesthetic and to the technical aspects. The early buildings, where analytical methods of stress distribution were very limited, were built using these physical models. They so clarified understanding that people had the courage to build. The built form so clearly understandable that the buildings are outstanding.

Models such as these which take up an equilibrium form are quite easily built. Models which will tell the engineer the stresses and deflections in the membrane under service conditions - wind and snow loading - are far more complex. Whilst there is still no practicable substitute for a carefully built aero-elastic model tested in a wind tunnel to understand the dynamic behaviour of a flexible structure, the use of computer modelling has become an indispensable and affordable way for tension structures to be understood by their designers. The pace of advance in use of these types of structure will only be set by the capacity of the individual architect, engineer or technician to master and use the necessary design methods efficiently. Most architects and engineers are not model makers but need codified methods.

In tension structures design using physical modelling there has always been a core model as a focus for all communication and decision making. Increasingly it is in a computer rather than on a table, and it is the achievement of this stage in the development of the design method which is being evolved now.

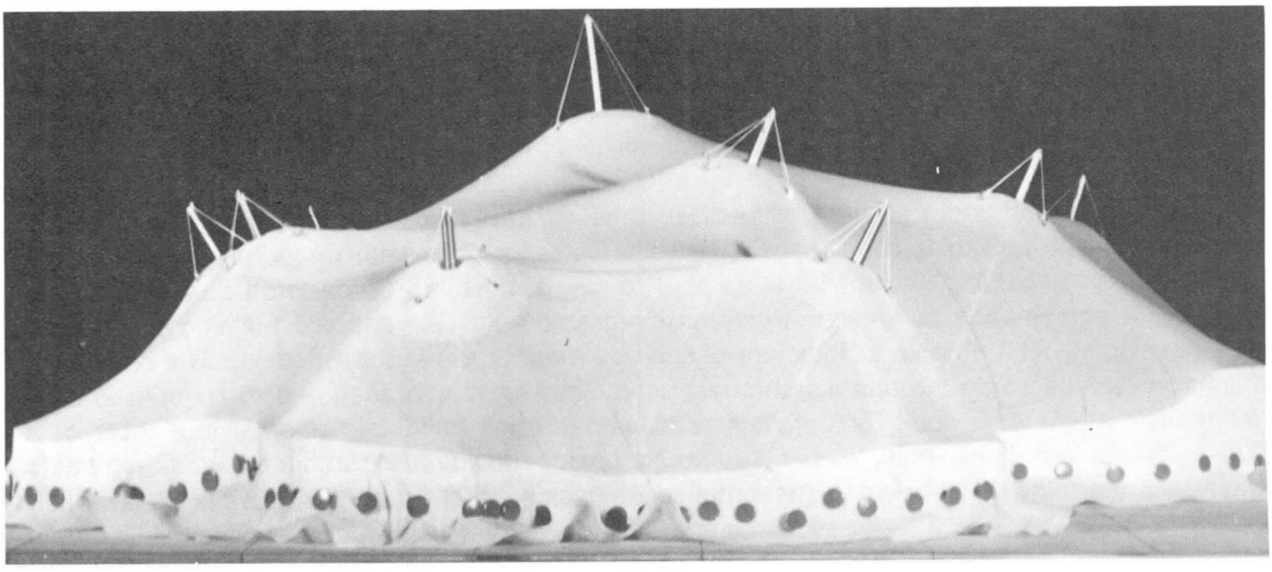

Figure 1: Use of physical modelling

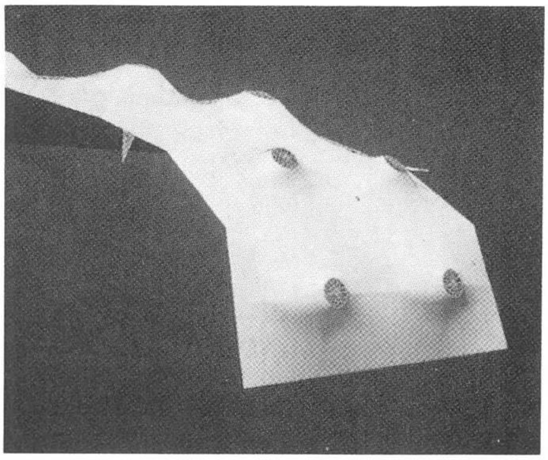

Figure 2: Use of computer modelling

Conclusion

The computer model now offers an ease and speed of interaction which has never been possible with physical models. For tension structures, where form is so closely allied to the structure's ability to perform, a model needs to be adjusted and "tuned" to give the optimum solution in all respects. Physical models using soap films or stretchy membranes are quite effective at providing an adjustable medium to assist decisions on final form. But they do not lend themselves to easy load analysis, accurate measurement or even walk through perspective views. The computer model offers quicker and more comprehensive interaction.

The existence of a single common numerical model does not guarantee free communication or successful design. The design process cannot be reduced to a series of self determining decisions and a single model is nothing if it cannot be both understood and used by those specialists in a design team for whom it has been "built". People need to know how to use the model and what the model will do for them. Different offices need to be able to work with the model simultaneously using their own familiar software. What matters is that all offices and therefore specialists use a common model which can serve all purposes.

It is interesting how the growth of knowledge has occurred. At first there developed a simple awareness of cables, then cable nets with cladding which had different stress-strain characteristics. Form finding techniques by modelling, how to determine loadings by wind tunnel testing and then strain loading wire models. The slow development of numerical analysis methods. Developments in membrane technology and an understanding of modes of failure. The importance of controlling the form and how economics is related to stress levels.

There are rich architectural possibilities for tension structures but the basic shapes are curved; they lead to sculpted buildings. There are associations with temporary buildings, but developments in cladding and membranes start to overcome this. Since large spaces are very economic a combination of tensile structures integrated with conventional building forms start to be developed. The organic architecture of such as Aalto provide study models. The complexity of light on surfaces, light through membranes, seam lines on membranes and so on start to send architecture students to look for parallels with historic shell buildings; after all the roofs of a mediaeval cathedral are only tensile structures upside down. So architects and engineers continually learn more to bring to the process of design. The scale of success is obviously related to how well we can interact our separate knowledge, experience and sensibilities. It must be through some form of "core modelling". Then not only are our possibilities extended but we can also develop more clearly the art of our time.

Sydney Aquatic Centre Roof Drainage Model Tests

Modèle du drainage des toitures du centre de sport aquatiques de Sydney

Modellversuche zur Dachdrainage von Sydneys Wassersportzentrum

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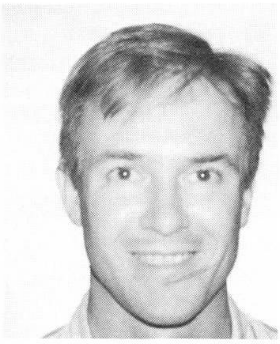

Simon Beecham, born in 1963, completed both undergraduate and doctoal degrees at the Univ. of Manchester, UK, where he was a lecturer before he moved to Australia in 1991. His current interests are in urban stormwater drainage and sewerage systems.

SUMMARY

This Aquatic Centre, designed for the Sydney 2000 Olympic Games, has a spectator seating capacity of 12' 500 and is covered by a complex roof configuration. A trapezoidal internal metal gutter collects storm water runoff from two roof surfaces (3' 800m2). Supported from above by an external transfer arch, the gutter is designed to convey runoff from a 100 year average recurrence interval storm. Vertical ties from the transfer arch protrude into the gutter floor to obstruct storm water flows. Design features of the roof and hydraulic model tests are described.

RÉSUMÉ

Ce centre de sports aquatiques fait part du complexe projeté pour l'Olympiade de 2000. Le centre offre 12'500 places assises. Il possède une toiture à structure porteuse compliquée. Un chêneau métallique trapézoïdal collecte l'eau de pluie provenant de deux pans de toiture (3'800 m2). Suspendu à un arc porteur extérieur, ce chêneau a été dimensionné pour absorber l'eau de la tempête maximale de 100 ans. Les suspentes de l'arc traversent le fond du chêneau. Les particularités du projet de la toiture et les études hydrauliques sur modèle sont décrites.

ZUSAMMENFASSUNG

Das Aquatic Centre wurde für die Olympiade 2000 mit 12'500 Sitzplätzen entworfen. Die Anlage ist durch ein kompliziertes Dachtragwerk überdeckt. Eine innere trapezförmige Wasserrinne leitet das Meteorwasser von zwei Dachflächen (3'800 m2) ab. Aufgehängt an einem externen Stützbogen, wurde die Rinne für einen 100jährigen Gewittersturm bemessen; die Hängestangen vom Bogen treten im Rinnenboden als Abflusshindernisse hervor. Entwurfsbesonderheiten des Daches und hydraulische Modellstudien werden vorgestellt.

1. INTRODUCTION

Friday 23rd September 1993 will always remain a memorable day for the sport-loving Australian public. For those associated with the Olympics 2000 engineering works the feeling of excitement was even more intense. Over the past two years a 760 ha derelict site at Homebush Bay, 15 km west of Sydney city centre, has been gradually transformed into a futuristic Olympic venue. In particular, innovative architectural, engineering and landscape works have been integrated in the design and construction of the Sydney International Athletic and Aquatic Centres. Most of the work has been project-managed on behalf of the New South Wales State Government by Civil and Civic Pty Ltd, a major Australian contractor. Figure 1 shows the location of the Homebush Bay site.

The original masterplan allowed for completion in either Olympic or non-Olympic mode. In the latter mode the Sydney International Athletic Centre has a 15,000 seating capacity competition arena and warm-up track. In Olympic mode the competition arena becomes a warm-up track and a new 80,000 seating capacity stadium will be built next door.

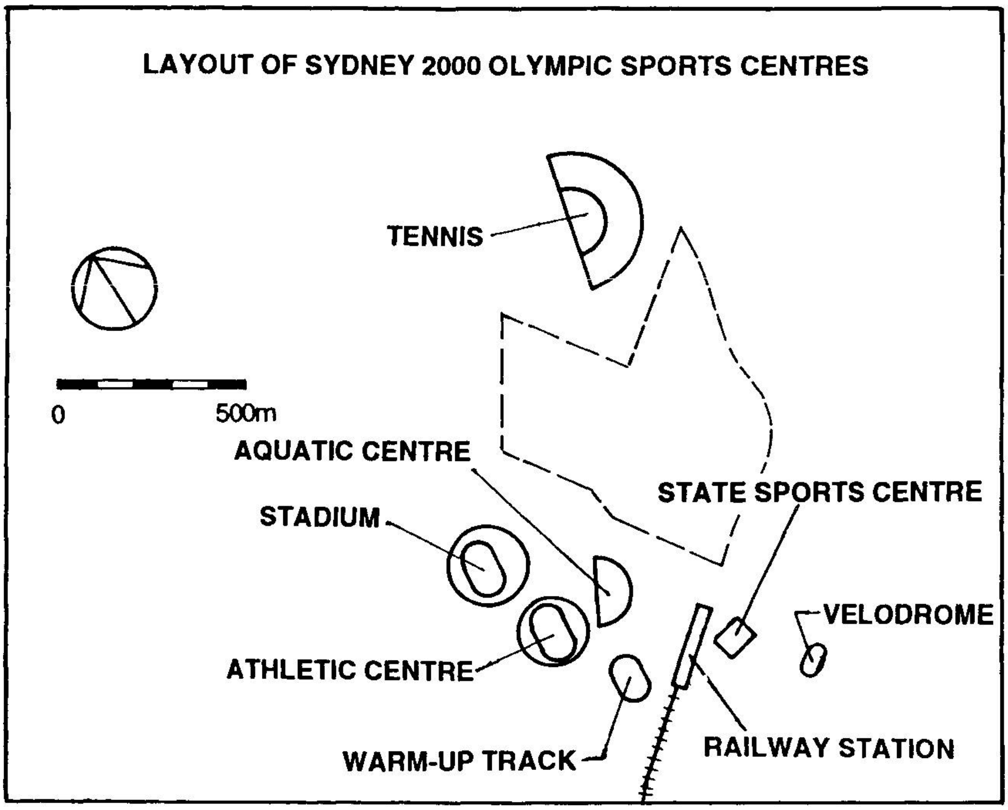

Fig.1 Location and Layout of the Homebush Bay Site

2. SYDNEY AQUATIC CENTRE

2.1 General

The Sydney Aquatic Centre has a non-Olympic seating capacity of 4,400. Additional undercover area will be provided for the Olympic Games, increasing the seating capacity to 12,500. The entire viewing area will be air-conditioned with outlets placed beneath or behind all rows of seating. The Aquatic Centre encompasses leisure pools, a training pool of adjustable depth, a main competition pool of adjustable length and a diving pool. The entire structure is enclosed with a complex roof geometry providing an unobstructed plan space of 120 m by 62 m. A clear height of 13 m is available at the underside of the centre of the roof trusses. In Olympic mode an additional plan width of 25 m is provided on one side.

2.2 Roof System

The main pool area is roofed with an arched truss arrangement. In non-Olympic mode the curved main roof has a plan area of 2,575 m^2. A monoslope roof of plan area 1225 m^2 is provided above the additional Olympic seating area. A line of sight from the uppermost row of seating to the far side of the main competition pool has to clear the junction of the underside of the main and monoslope roof areas. A trapezoidal internal gutter is provided to permit this line of sight. See Figure 2.

The trapezoidal gutter and roof junction are supported from above by an external transfer arch. This arch is constructed of uniform radius steel box section with steel ties supporting the gutter, the curved main roof and the monoslope Olympic extension. The total roof weight is approximately 640 tonnes. The box section arch below the gutter bifurcates, providing both an aesthetically attractive feature and additional lateral stability.

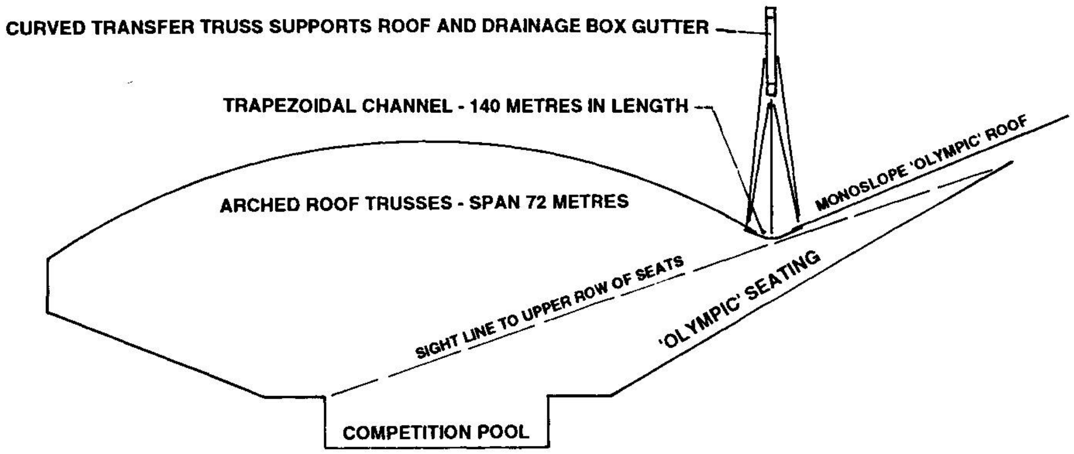

Fig.2 Sydney Aquatic Centre, Elevation View

3. ROOF DRAINAGE

The trapezoidal gutter is supported from above by vertically oriented ties which are streamlined with aerofoil-shaped casings. This design aims to minimise disturbance to the water flow during storm events. An outlet is located at each end of the 140 m long gutter. Each outlet has a steep sloping open channel section, allowing rapid freefall discharge of the stormwater flow.

The large, unusual shaped gutter is designed to convey a 100 year average recurrence interval (ARI) storm. During a storm event the protruding streamlined support tie casings will present obstructions to the flow. The 100 year ARI storm runoff corresponds to flows into the gutter of approximately 3.2 L/s/metre width for the arched roof alone and 4.7 L/s/metre width for the arched and monoslope roofs combined. These are referred to in the model testing as the *low* and *high* flow rates respectively. Note that only a single roof surface was used in the model tests.

Water flowing laterally into a longitudinal open channel flow produces a complex hydraulic regime known as *spatially varied flow.* Current Standards Association of Australia [1] guidelines for the design of internal gutters are based on work conducted by Martin [2] and Martin and Tilley [3]. The analysis of spatially varied flow for roof drainage design is described by Mein and Jones [4] and by Jones and O'Loughlin [5]. Analysis requires the prior knowledge of the effective roughness of the gutter. The flow is termed spatially varied because the flowrate varies along the spatial length of the channel. A complete theoretical analysis is not possible for the Sydney Aquatic Centre gutter because of the disturbances arising from the tie protrusions.

The Project Managers for the Aquatic Centre, Civil and Civic Pty Ltd, commissioned the University of Technology, Sydney (UTS) to conduct model testing in order to establish whether the unique gutter shape, and unorthodox overhead support structure could safely collect and discharge runoff from the design storm. UTS were at the time conducting full-scale experimental roof drainage tests for the Standards Association of Australia. These tests are described by Beecham and O'Loughlin [6].

4. HYDRAULIC MODEL

For a design rainfall intensity of 313 mm/hour, the model was required to estimate the depth of flow on the roof sheeting and the maximum depth of flow in the trapezoidal gutter, assuming a free outfall. The effect of tie protrusions on the maximum gutter depth was also investigated. The 9 m long model gutter was built full-scale (1:1). The model roof and gutter configurations are shown in Figure 3. It was not possible to model an arched roof in the laboratory. To obtain a conservatively high estimate of prototype gutter depth a plane roof was used, inclined at the maximum slope of the arched structure. One disadvantage with this configuration was the likelihood of underestimating the maximum water depth on the roof sheeting. This was not considered critical in light of the very shallow depths measured on the model roof sheeting.

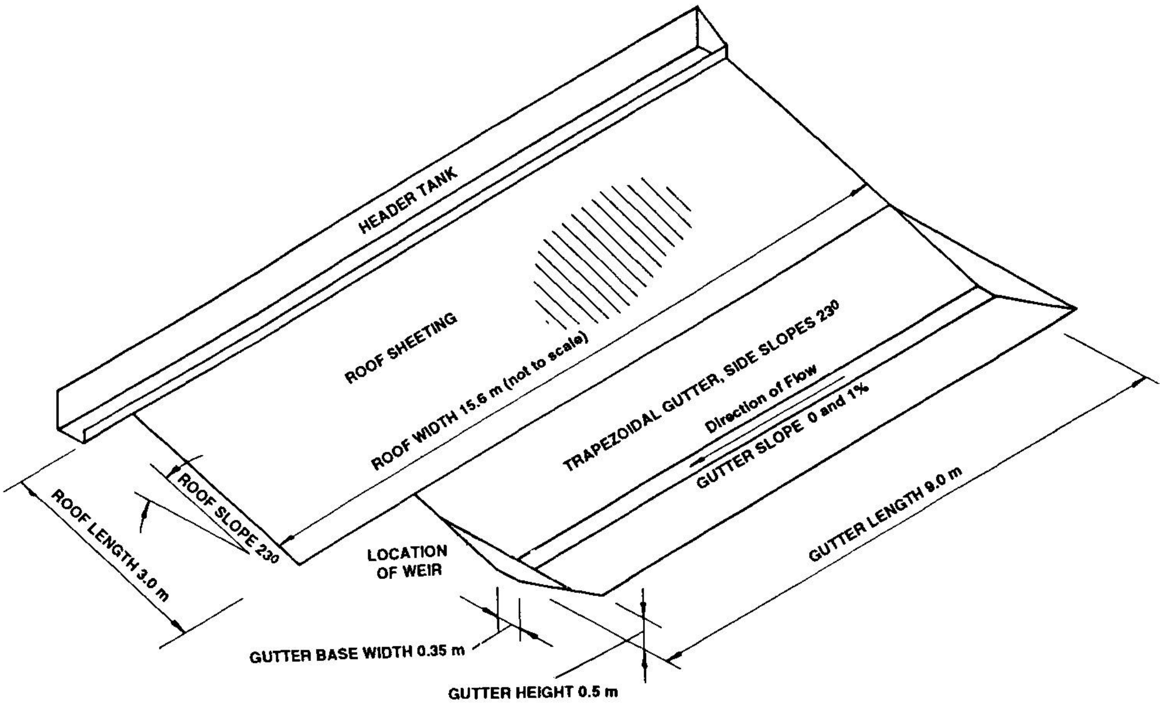

Fig.3 Model Roof and Gutter Configuration

Depth profiles were measured in the horizontal trapezoidal gutter for both the low (3.2 L/s/metre width) and high (4.7 L/s/m width) flowrates. The flows were supplied using a header tank suspended along the full length of the model roof, shown in Figure 3. Two further tests were conducted using the same gutter with a 1% fall. All leakage losses were collected and passed through a calibrated V-notch weir. For all tests the losses were found to be less than 0.1% of the flow rate. In both tests a free outfall was assumed at the downstream end of the prototype gutter. To model the effects of half the 140 m long prototype gutter, a transverse sharp-crested weir, with a sill height of 379 mm, was constructed at the downstream end of the model gutter. To ensure that the weir had no end effects on the measured flows, the roof discharge was restricted to the upstream 5 m of the model gutter, located 4 m to 9 m from the weir. It was calculated that this weir generated depths of flow in the model equal to those in the prototype gutter. Other assumptions were that rainfall impact and wind turbulence would not significantly affect the flows, and that the roof flow was fully established over a three metre length. Each roof and gutter configuration was tested for the effect of a tie member protrusion located at various positions along the gutter.

5. RESULTS

5.1 Depth of Flow on Roof Surface

The depth of flow on the model roof was independent of the gutter configuration, and was only influenced by the flow rate. Figure 4 shows the average measured depths of flow in the Alcan LT7 roof sheeting pans for the two flow rates. It was concluded that, for the steepest roof slope of 23° the sheeting profile was adequate to convey the design flowrates. Note that it was not possible to model raindrop impact and wind effects.

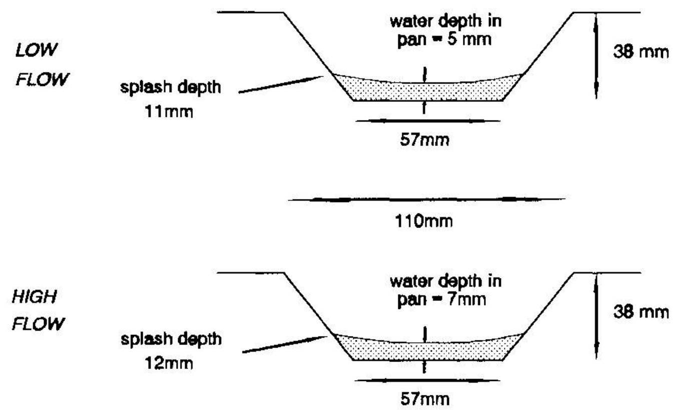

Fig.4 Depths of Flow on Roof Sheeting

5.2 Gutter Depth Profiles

Average depths of flow, relative to the channel floor, were measured at 0.5 m intervals over the full 9 m gutter length. The lateral inflow from the roof surface was restricted to the upstream 5 m of the gutter (corresponding to the range 4 to 9 m from the weir on the following graphs). Only the depths in this 5 m length are indicative of the prototype depths.

For a horizontal gutter, the maximum measured depth of flow was 417 mm (see Figure 5) which occurred with a tie protrusion located 6 to 7 m from the downstream weir. For a gutter with a 1% fall, the maximum measured depth of flow was 375 mm (see Figure 6). This occurred with a tie protrusion located 4 to 5 m from the downstream weir. The flows are spatially-varied and the location of the maximum depth will alter with gutter slope. Therefore it is not possible to directly interpolate these results for gutter slopes between 0 and 1%.

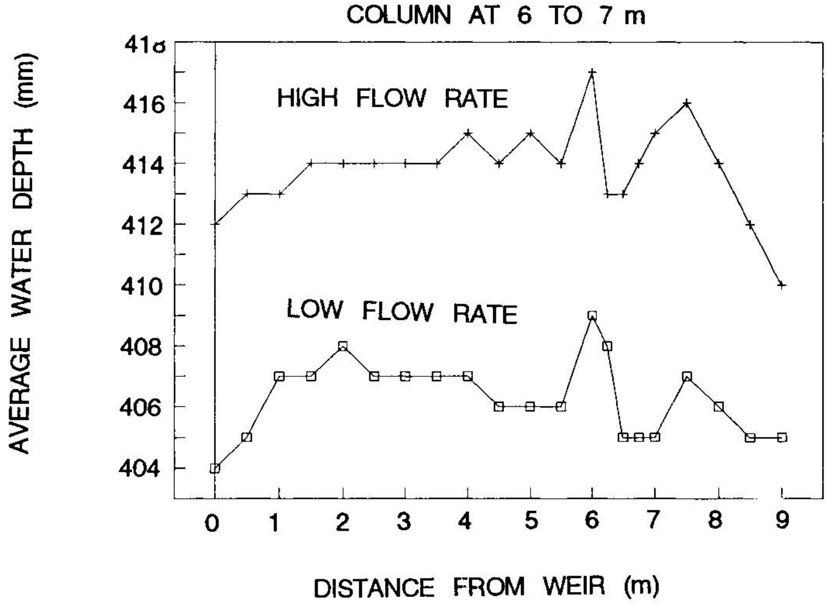

Fig.5 Horizontal Gutter

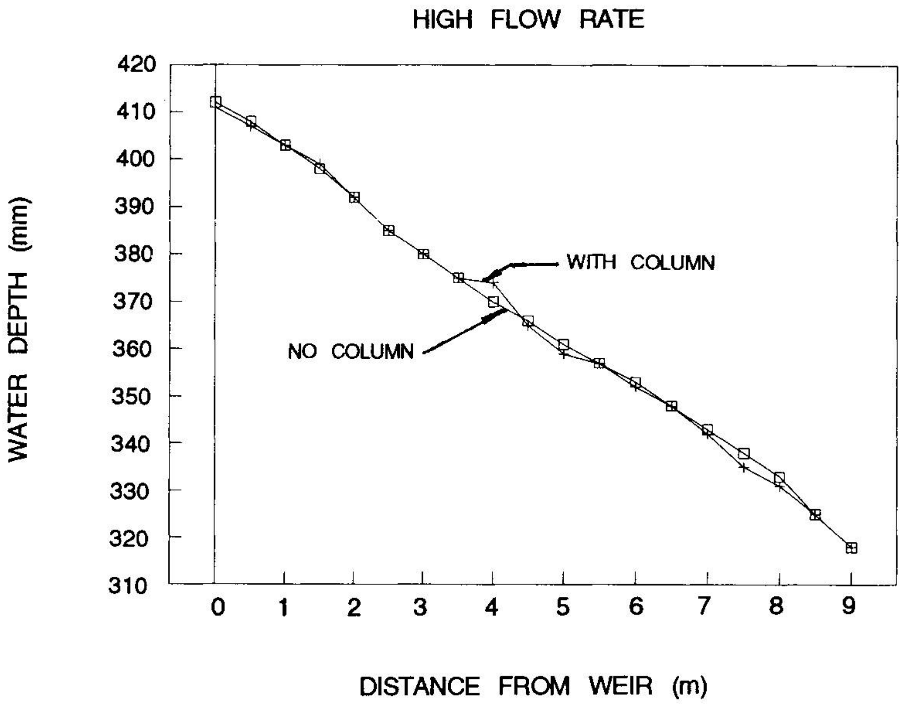

Fig.6 Sloping Gutter (1%)

6. CONCLUSIONS

From the results of the model testing program both the Alcan LT7 roof sheeting profile and the wide trapezoidal gutter designed for the Sydney Aquatic Centre have sufficient capacity to safely drain the roof runoff corresponding to a 100 year average recurrence interval storm. The streamlined casings of the tie protrusions from the overhead support truss do not significantly affect the water depths in the trapezoidal gutter.

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Symphony Hall Birmingham - Railway Isolation and its Maintenance

Salle de musique Birmingham - Isolation et maintenance

Konzerthalle Birmingham - Schwingungsisolierung und Unterhaltung

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R. Greenwood's project experience includes the OAU Conference Centre in Liberia, St. David's Concert Hall in Cardiff and the Sir Adrian Boult Hall in Birmingham. He was the engineering design project director for the International Convention Centre and Symphony Hall, Birmingham.

Alan CARNEY
Engineering Manager
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Alan Carney gained his initial engineering experience in the aerospace and manufacturing industries. Since 1989 he has been the Engineering Manager initially for the International Convention Centre and on completion of the National Indoor Arena, responsible for both the ICC and NIA.

SUMMARY

The City of Birmingham's brief for the International Convention Centre and its Symphony Hall required that the facilities should rival the best in the world. The City Centre site is bisected by a main line railway tunnel. The project achievements included reducing the vibration from the railway to an extent which tested the limits of technology and workmanship. This paper considers the design of the railway vibration isolation system and its maintenance.

RÉSUMÉ

Le mémorandum de la ville de Birmingham pour l'édification de l'International Convention Centre avec salle de concerts indiquait que les aménagements devaient être parmi les meilleurs du monde. Or une importante ligne de chemin de fer passe en tunnel sous la zone à bâtir. Une partie de l'étude avait pour objet de réduire les vibrations dues au passage des trains à des valeurs pouvant atteindre les limites de faisabilité aussi bien techniques que professionnelles. Les auteurs présentent le projet et la maintenance du système d'isolation antivibratoire.

ZUSAMMENFASSUNG

Die Bestimmungen der Stadt Birmingham für das International Convention Centre mit Konzerthalle verfügten, dass die Einrichtungen der besten der Welt gleichkommen sollten. Der Tunnel einer wichtigen Eisenbahnlinie durchquert das Baugelände. Ein Teil der Projektleistung bestand darin, die Erschütterung durch Züge in einem Masse zu reduzieren, das zu den Grenzen des technisch und handwerklich Machbaren vorstiess. Der Beitrag behandelt Projektierung und Unterhaltung des Systems zur Schwingungsisolierung.

1. THE DEVELOPMENT

The Centre includes the 2,200 seat Symphony Hall and two Convention Halls of 1,500 and 300 seats respectively, each with raking floors. There are two flat-floored halls, of 2,700m² and 900m² respectively, that can be used for banqueting, exhibitions or 'pop' concerts. Six multi-purpose break-out and seminar rooms complete the facilities with associated registration, foyer, catering and support spaces.

Strategic planning began by locating the noise critical spaces away from the tunnel and its low frequency ground borne energy. No single means of attenuation could achieve the targets, so in addition to isolating the rail track, the design included heavy piled foundations with the top of the pile isolated from the ground. Further the noise critical halls were floated on elastomeric rubber bearings and were separated from the other buildings by jointing systems through both services and structures to avoid lateral vibrations.

Demanding construction procedures and inspection methods were overseen by a Management Contractor who was responsible for maintaining the integrity of the vibration isolation throughout the construction and installation phases. For optimum attenuation, each rubber bearing had to be tested and certified at works. Construction had to be planned to ensure even load transfer and each set of bearings had to be monitored to avoid under or over compression becoming a source of potential vibration bridging.

The acoustic performance is monitored through annual visual inspections of the railway track isolation and annual repeats of the acoustic tests to identify any degradation and its likely source.

The Symphony Hall has been acclaimed by performers and critics alike for its acoustic performance.

1.1 Location

The two and a half hectare city centre site for the International Convention Centre is bisected by the Monument Lane Railway Tunnel. This brick lined, twin track tunnel was cut in 1854 by Robert Stephenson. It now carries Inter-City trains from London Euston northwards via Birmingham New Street. The influence of the railway tunnel and the relative compactness of the site for 50,000m² development have both had fundamental effects on the planning arrangement for the eleven hall complex, see Figure 1.

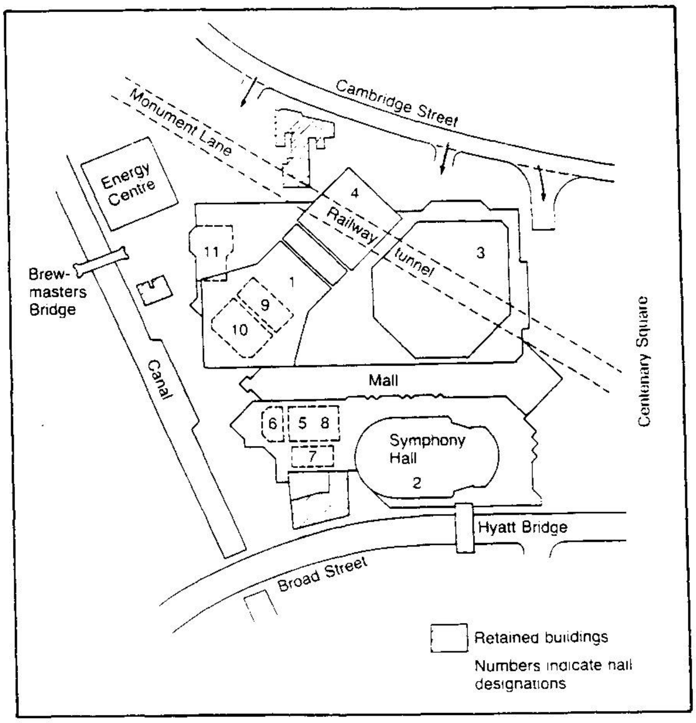

Fig.1 Plan: The Symphony Hall is located 35m from railway

2. RAILWAY VIBRATION

2.1 Vibration Investigation

Two surveys were carried out to determine vibration levels at various locations on and adjacent to the site.

The first concentrated on velocity measurements on the surface. One particular feature from the first survey was the concentration of energy in the 63Hz octave band. This arose because of the spectral emphasis within the train vibration. Results for a typical train are shown in Figure 2.

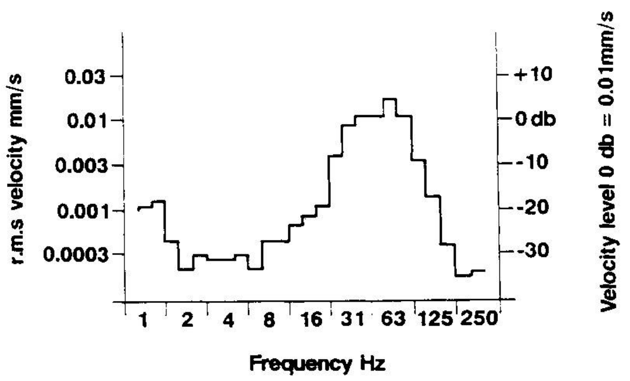

Fig.2 Typical ground vibration spectrum: measurements at 50m from tunnel.

A second survey was arranged to improve understanding of the specific pattern of vibration propagation from this tunnel. A methodology was developed for vibration measurements within boreholes. Accelerometers were placed in them to sample ground response arising from compression and shear waves.

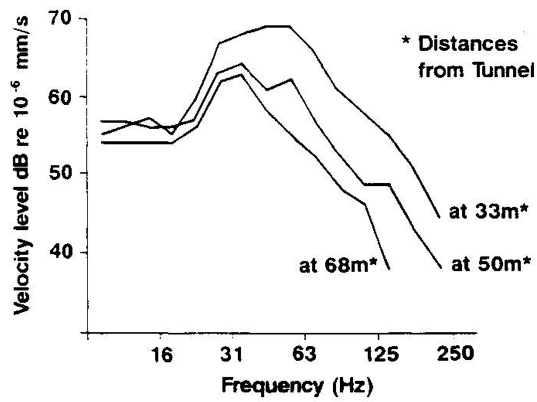

Fig.3 Borehole vibration data: Vibration at 33m, 50m and 68m from tunnel.

Examples of vibration velocity data from three boreholes are given in Figure 3. Although horizontal vibration levels were significant, the vertical component was found to dominate.

2.2 Planning Considerations

To achieve the number and size of halls on the constrained site, many of the halls had to be stacked either above each other or over support service space. A strategic approach had to be followed in order to protect the noise critical spaces, particularly in the Symphony Hall, Hall 1 and Hall 5. Substantial isolation at source, ie. at rail track level, was not practical.

2.3 Acoustic Targets

The Acoustic Consultant called for design work to be directed at the achievement of inaudibility of trains in the Symphony Hall. This demanding target meant that the low frequency ground borne vibration had to be attenuated and prevented from becoming significant structure borne energy that would result in noise.

3. ISOLATION AND ITS MAINTENANCE

The approach to isolate the halls from the low frequency ground borne vibration was to combine the contributions from seven different means. These are discussed below:-

3.1 Partial Track Isolation

Early discussions with Civil Engineers from British Rail revealed that although major track level isolation was impractical, they had plans as a part of their cyclic maintenance plan to relay the track and sleepers in the tunnel during the construction period of the Centre. This offered a chance for under-sleeper isolation to be added. A sheet material manufactured by James Walker Ltd, which had been on trial in North Wales, had provided close to 10dB attenuation in the 63Hz octave band. It was felt that perhaps 6dB to 8dB might be achieved in the tunnel. In view of the wish to minimise transmission to the site, it was agreed that the tracks should be isolated. On site measurements have confirmed that a useful benefit has been achieved and Figure 4 shows results from a test train before and after installation beneath one of the lines. This installation provided a general, site wide lowering of vibration levels.

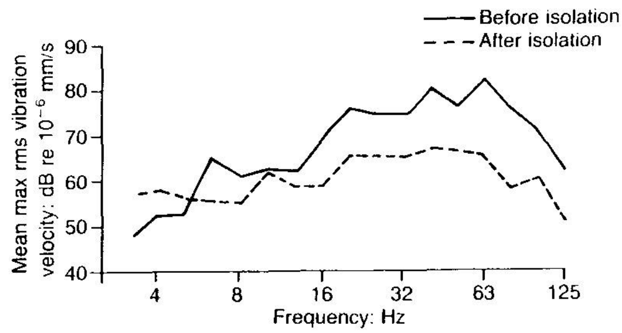

Fig.4 Rail isolation test train results: vibration velocity before and after isolation of rail sleepers.

It has been agreed with the Railways Board as part of the ongoing monitoring of the acoustic performance of Symphony Hall that periodic inspections of the track isolation will be allowed with the proper prior notice and safety provisions. These are jointly actioned by ICC Engineering and Ove Arup.

3.2 Distance

The Symphony Hall, being the most sensitive and noise critical space of the development, had to be located in the most favourable part of the site, ie. where the least ground borne energy existed. It was only possible to achieve just 35m to the Symphony Hall auditorium. At this distance the energy was beginning to decay and had become dominated by the surface wave.

3.3 Foundation Type

To maximise the attenuation, 'heavy' foundations were planned to support the noise critical halls remote from the tunnel. Foundations were arranged to take support only from the least mobile rocksand.

3.4 Pile Form

Large diameter bored piles cast into the Rocksand satisfied the 'heavy' foundation requirements. It was also possible to decouple the top of the Symphony Hall piles from the surface wave. Six metres was determined as a maximum realistic depth for the decoupling. Many materials were considered as sleeves at the top of piles to provide a mismatch against vibration coupling. To achieve the benefit, sleeving material needed to be highly compliant. It was eventually decided to use an air void around the top of the 120 piles needed to support the Symphony Hall. The piles were formed using concentric steel tubes as permanent formwork.

The pile boring was then advanced by continuing drilling down through the inner steel tube to form the rock socket. The pile and substructure arrangement is shown in Figure 5.

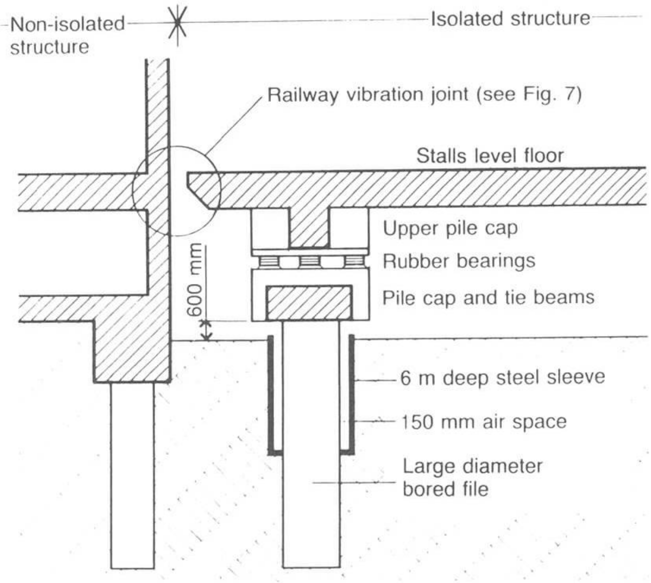

Fig.5 Isolated Symphony Hall foundations

Protective collars are in place to prevent the inadvertent introduction of any waste material into the void. Periodic inspections check the voids for the ingress of foreign material or water. Automatic leak detection alerts Engineering Staff to problems in the undercroft through the Building Management System.

3.5 Substructure

For the most noise critical halls, isolated superstructures are floated on twin layers of foundations, separated by a layer of bearings. Lower sets of beams restrain pile caps and they are always 600mm above the ground to maintain the integrity of ground isolation. Upper beam arrangements collect the superstructure loads together, see Figure 6.

Tight control on the geometry of air spaces is needed because, the stiffness of large expanses of shallow air space can lift the frequency. Also, the build up of resonances in voids can aid transmission sufficiently to have adverse effects on isolation. The complex geometry prevents powerful resonances occurring. The area of pile caps is limited to avoid excessive local coupling through air across the separating isolation bearings.

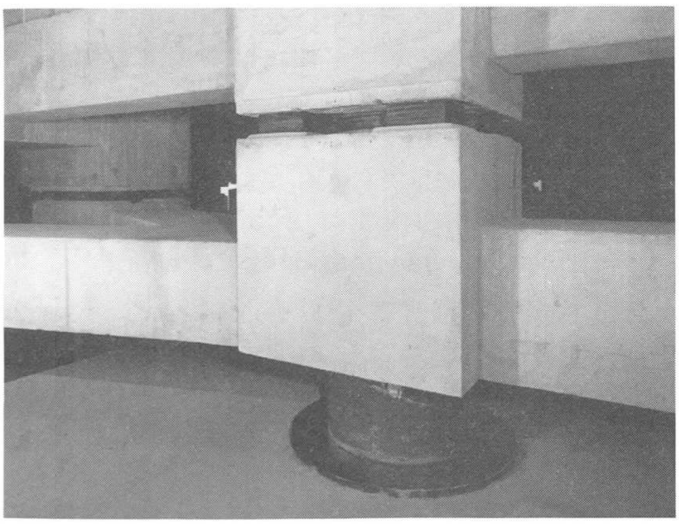

Fig 6. Symphony Hall foundations.

The integrity of the air space in the undercroft is protected both for acoustic requirements and for fire loading control. This requires that the undercroft remains a sterile area with the minimum of services and the prohibition of use for storage.

3.6 Bearings

Bearings separate isolated superstructures from their substructures. The bearings are steel plate-reinforced, natural rubber-based compounds manufactured and supplied by the Andre division of BTR Silvertown. A full programme of static and dynamic testing has been carried out, all bearings are numbered with individual static test certificates. Creep, ozonization, shear and durability assessments have followed the guidelines of BS6177: 1982.

Bearings are set onto levelled epoxy grout beds around a steel failsafe block. The construction followed the setting of the bearings on the epoxy and involved each array of bearings having a temporary edge shutter to allow the space between them to be filled with sand. A 50mm mass concrete slab the same size as the pile caps forms a permanent shutter to the upper cap. Each bearing array was monitored during construction. The mass concrete permanent shutter was also deemed to be a sacrificial slab should any bearing need removing or replacing. In fact, a few bearings were removed during commissioning to increase compression on some arrays.

3.7 Separation Joints

Where either isolated or non-isolated structure meet floated structures, a 50mm air space has been allowed. Total relative vertical settlement of as much as 12mm has been allowed across railway vibration isolation joints (RVJ's). Typically, joints have been designed to accommodate ± 6mm of lateral movement.

The elevational area facing the floated structure has to be limited, slabs and walls are generally chamfered to a 75mm deep nosing. A series of special bridging details deals with the need to carry people over the joint, achieve fire separation, and account for settlement and bearing creep without vibration bridging. Folded foam or ceramic blanket is used to achieve fire ratings within isolation joints, see Figure 7. For waterproofing or facing, low modulus sealants or dry resilient seals have been included.

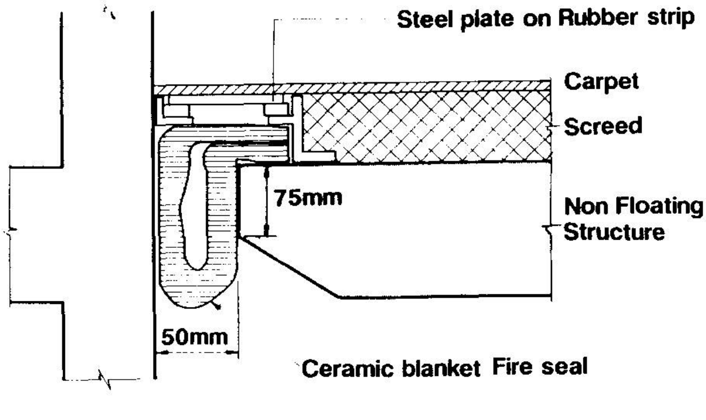

Fig.7 Railway vibration joint: isolation gaps separate all halls.

To ensure the original specification is protected through the engineering disciplines applied that prohibit the bridging of acoustic joints by rigid services runs during maintenance or modification work through a system of work permits and supervision.

4. CONCLUSION

Reliable measurements of any residual train noise are not realistic in relation to the very low levels of ambient sounds in the Symphony Hall. This underlines effectively the success of the isolation procedures and their continuing maintenance.

Inspection and Certification of Passenger-Carrying Rides

Inspection et autorisation d'exploitation des transports de personnes

Inspektion und Zulassung von Passagiere befördernden Bahnen

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John Roberts, born 1948, obtained his BEng from Univ. of Sheffield. He has worked on a wide variety of civil and structural engineering projects and has been involved in theme park rides for the past eight years. He is an elected member of Council of the Institution of Structural Engineers and currently serves as Honorary Treasurer and Chairman of the Adjudicating Panel.

SUMMARY

This paper describes and comments on the voluntary UK scheme for inspecting and certifying theme park and fairground rides, and makes particular reference to the public safety aspects related to structural safety of the rides themselves.

RÉSUMÉ

L'auteur décrit et commente le programme britannique non obligatoire sur l'inspection et l'autorisation d'exploitation de voies ferrées dans les foires et parcs d'attractions. Il attire l'attention sur les aspects de la sécurité publique qui est directement liée à la sécurité des structures porteuses des voies de transport elles-mêmes.

ZUSAMMENFASSUNG

Der Beitrag beschreibt und kommentiert das freiwillige britische Programm zur Inspektion und Zulassung von Bahnen in Vergnügungsparks und auf Rummelplätzen. Insbesondere wird auf die Aspekte der öffentlichen Sicherheit hingewiesen, die mit der Tragwerkssicherheit der Bahnen selbst verbunden sind.

1. INTRODUCTION

Recent estimates show that more than 500 million individual rides are enjoyed each year in the U.K. on the 10,000 or so passenger-carrying amusement devices that operate on theme parks, fairgrounds and travelling shows. Some large-throughput rides at fixed locations have annual usage some of the order of 1,500,000 persons per year. Typically, these rides will be used on a more-or-less continuous basis for a season of 1,500 hours spread over the period from late March to early November and thus have an average throughput of 1,000 persons per hour. Visitor numbers exceed 6.5 million per year at the largest U.K. site (Blackpool Pleasure Beach) where entry is free and a charge levied for each ride, and numbers exceed 2.25 million at the largest U.K. site where a single entrance fee is charged (Alton Towers).

Notwithstanding periodic media interest in accidents on rides, a study undertaken for The Health & Safety Executive in 1990[1] concluded that the risk of serious injury or death during an assumed typical visit ("session") by a member of the public was significantly safer than the risks of:

Travelling to the fairground by car
Cycling (for the same time as actually spent riding rides)
Horse-riding (for the same time as the "session")

by a factor between 7 and 20 times safer. Somewhat surprisingly the analysis also shows that it was about twice as safe to spend a two-hour "session" at a fairground, riding 10 rides for a total of 20 minutes compared to 2 hours of "average existence" outside a fairground.

The causes of accidents that do occur on rides can be classified broadly into two classes: Structural and Operational. Structural causes include malfunction and physical failure of the ride itself and can be conveniently subdivided into structural, mechanical and electrical safety, the latter including, for certain major installations, instrumentation and control. Operational causes includes accidents caused by operator or attendant acts or omissions and also direct passenger or other visitor/public failure to observe safe behavioural practices. For the purposes of the remainder of this paper, it is intended to concentrate on the Structural aspects of ride safety (using the wide definition of Structural given above, i.e. including mechanical and electrical safety).

2. HISTORICAL REVIEW

Fairground rides are believed to have originated in Russia in the eighteenth century, when artificial ice-slides were constructed from timber structures and passengers rode the slopes on toboggans. Wheeled versions (also on wooden, but not "ice" slopes) were introduced into France and spread rapidly into a number of European countries.

The birth of modern fairground rides occurred in the last two decades of the nineteenth century in the U.S.A. which has since consistently led the world in more sophisticated and more thrilling ride developments. In the U.K. the influence of the U.S.A. has always been strong and fixed fairgrounds based on American rides and attractions were established here before 1900.

By the 1930's very major rollercoasters had been constructed in the U.K. (a number of which survive reasonably intact from that time) and in the 1960's and a new breed of steel rollercoasters began to appear. In 1978, for example, Blackpool Pleasure Beach installed Europe's "first" looping coaster (the Revolution) which was made in America, and by the 1980's an explosion of new rides, new theme parks and revived interest in their enjoyment was due partly to increasingly easy access to the U.S.A. (particularly to Disneyworld in Florida) and partly to the success of "greenfield" U.K. locations like Alton Towers and Thorpe Park.

Although there have always been accidents on fairground rides since they were first introduced, public attention in the U.K. was particularly focused by the major roller-coaster accident at Battersea Fun Fair in 1972 when a mechanical failure led to the deaths of five children. Since then many of the more reputable owners/operators have made attempts to conduct independent inspections of their rides; operator/mechanical inspections had always been undertaken, but it is probably fair to say that they had been directly concerned with the requirement to operate the ride without malfunction or breakdown rather than explicitly for safety-related purposes.

In 1984 The Health and Safety Executive (HSE) introduced the "Code of Safe Practice at Fairs"[2] which provided a framework for annual independent inspections and certification of all passenger-carrying devices. It was, and remains to this day, a voluntary scheme, although the main trade bodies of theme parks and fairgrounds (BALPPA and The Showmans Guild) are understood to make compliance with the Code a condition of membership. In the case of BALPPA, this extends to an obligation to file annual copies of the Report of the Thorough Examination and Certificate (if issued) with BALPPA, which also maintains an index of unique ride identification numbers that are particularly useful when rides are bought and sold and details of their past history are being reviewed.

To provide further and more detailed technical information on the design, construction, modification, repair and maintenance of rides, a "Technical Annex" to the Code was published in 1988[3]. A revised version of the code itself, "Fairgrounds and Amusement Parks : A Code of Safe Practice" was published by HSE in 1992[4]. It is worth noting that both editions of the code and technical annex have been written with their active participation of the industry itself.

The new code (1992) includes a requirement for a Functional Test, on an annual basis, as part of the Thorough Examination. Previously, tests were only required every 4-years and understandably there was concern that some examiners may have issued certificates for rides which they had not observed in operational mode.

3. DESCRIPTION OF THE PRESENT U.K.SCHEME

Inspections, called "Thorough Examinations", are carried out on each ride, on an annual basis, by a "competent person". There is no specific requirement as to the qualifications of the competent person, although the code does require them to have appropriate "qualifications, knowledge, experience and supporting resources". The only specific requirements set down in the HSE code are that the person shall be at least 25 years old, and shall be independent of the operator/owner of the ride. Most ride examiners are members of a recently established body called (NAFLIC, National Association for Leisure Industry Certification) who are currently drawing up guidelines for the qualification levels considered appropriate for individual "competent persons" but naturally these cannot be mandatory in the present regulatory framework.

Practice varies as regards the different aspects of the inspection, i.e. the structural, mechanical and electrical safety considerations. For small devices, typically non-electrical or simple motor-driven children's rides, one person frequently carries out all three items. However, some inspection organisations prefer to combine the structural and mechanical aspects (which are often very closely linked in any event), and then have all electrical inspections carried out separately, by a different person, who will issue a separate report and certificate. For very major rides, e.g. purpose designed rollercoasters, three separate persons with the necessary skills and expertise for each discipline may well be involved. Careful coordination of the physical inspections and the reports is required to ensure that the owner/operators interests are properly attended to.

Practice also varies as regards timing of the inspections. Almost all rides in the U.K. are operated on a seasonal basis for between 200 and 240 days (March/April to October/November) and the close season is obviously, for commercial reasons, when most inspection work is undertaken. Almost all rides require dismantling to some extent to allow inspection and then reassembling for the functional test run. Inspections are commonly done either soon after the season ends, when rides are stripped down and the inspection reports form a basis for any maintenance and repair/replacement work required; or alternatively towards the end of the close season when all the routine maintenance etc has been done by the owner/operator. Each of these approaches has both merit and disadvantages as follows:

Early inspection - gives the owner/operator notice of repairs/replacement in good time, but may encourage the use of the inspections as a guide for maintenance/repair and so lessen the responsibility of the owner/operator to establish and undertake this work to his own satisfaction. Also, if faults or defects are found, the inspector will need to make a return visit after repair works have been completed, but before reassembly, to re-inspect the ride in order to issue certification.

Late inspection - allows the owner/operator to present a ride after all maintenance/repair work has been completed, matching more closely the intention of the independent inspection requirements. However, if any defects or faults are discovered by the inspector, there may be severe pressures on time for the owner/operator to carry out work and have it re-inspected.

The issue of a certificate is not made using a fully standardised format, although Appendix 2 of the HSE code does specify key features of the content of the certificate. Of particular importance is to clarify whether the certificate covers only one or more of the categories, structural, mechanical and electrical and to precisely specify any work needed on the ride within specified time or usage limits (work needed before use of the ride would normally prevent issue of a certificate to use the ride, as re-inspection would be necessary).

During both structural and mechanical Thorough Examinations it is often necessary for NDT inspection techniques to be used on steel fabrications, which are susceptible to fatigue cracking from the repeated loadings associated with dynamic ride motions. It is very important however, to use NDT as a tool of the appointed person carrying out the examination, and not to rely wholly on the technology of the NDT work itself. The Code of Safe Practice acknowledges this (clause 101) by placing an obligation on the appointed person to specify to an NDT operator where tests are to be undertaken.

4.THOROUGH EXAMINATION PROCEEDURES

The use of the term "Thorough Examination" has been deliberate, so as to align the standard of work with the particular meaning of the term that has grown up by custom and practice in other safety-related industries such as pressure vessels and cranes.

At the core of the procedure is a careful and detailed visual examination to check for wear, signs of misuse or distress, misalignment, lack of fit or other defects. A clear understanding of how the ride operates is necessary in order to evaluate both the most likely locations for defects to occur, and the most critical components where defects or failure could have serious consequences.

Considerable guidance is available from experience on common rides (either identical production runs or similar variants) and aside from any individual examiners own expertise, two published sources of advice and information are Oas follows:

A series of Guidance Notes published by HSE.
A series of Technical Bulletins issued by NAFLIC for examiners.

Many rides have multiple identical components, for example, suspension arms on rotating "twister" type rides. Here a suitable strategy is normally to select a sample of these components for examination, maintaining a detailed record of which components were examined from one year to the next so that appropriate sampling coverage is provided.

Prior to the actual physical examination data needs to be collected on the available information from the manufacturer and operator. Of great importance is information relating to modification and repair work; operators appear to be very pragmatic in relation to undertaking such works and much practical work that in other industries would be carried out by specialist external contractors have traditionally been carried out in-house by the operators own workshops and staff. With some notable exceptions, the quality of records of such work has not matched the quality of the work itself, which causes problems in establishing exactly what has been changed or repaired.

An excellent written flowchart of examination procedure is provided in the Technical Annex which combines structural and mechanical inspection into one sequence, and then provides a separate sequence for electrical inspections.

5. CONCLUDING REMARKS

The U.K. voluntary scheme of ride inspections and certification has now been in operation for eight seasons and has been very widely taken up and accepted by operators and owners. Considerable maturity has been achieved with the active cooperation of the industry and the author is convinced that both overall standards and expectations of safety for the public in using passenger-carrying fairground rides has been enhanced.

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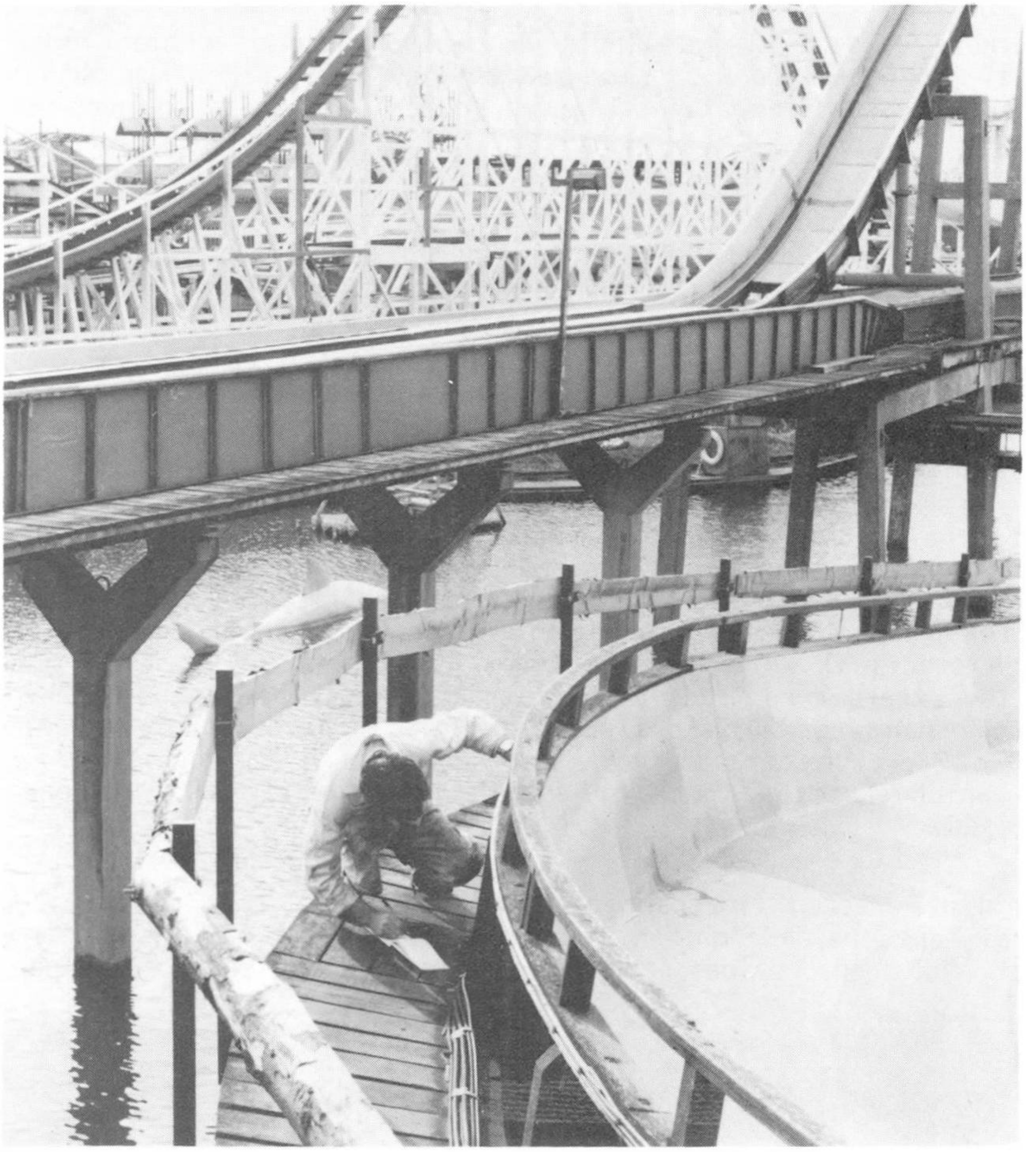

Engineering Maintenance at Birmingham International Airport

Organisation de la maintenance de l'aéroport international de Birmingham

Unterhaltungsplanung für Birminghams Internationalen Flughafen

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Tony Hartley, born 1944, qualified as a Chartered Municipal and Civil Engineer before joining West Midlands County Council and leading the design team on the 1977 - 1984 Airport Project. As Head of Engineering, he had the liaison role during the Eurohub Project. He has been General Manager of Engineering Services since 1989.

SUMMARY

The paper demonstrates the approach to maintenance at Birmingham International Airport since the opening of the Main Terminal in 1984 and Eurohub in July 1991. It reflects the changes introduced to accommodate new buildings and increased passenger throughput as well as a move from Local Government practice to an independent Public Limited Company.

RÉSUMÉ

La communication expose la procédure prévue pour la maintenance de l'aéroport international de Birmingham, depuis l'ouverture du terminal principal en 1984 et de l'Eurohub en 1991. Elle reflète les modifications rendues nécessaires, tant par l'intégration de nouveaux bâtiments et l'accroissement du flux de passagers que par la remise de l'exploitation, de l'administration publique locale à une société indépendante.

ZUSAMMENFASSUNG

Der Beitrag beschreibt die Vorgehensweise für die Unterhaltung des Internationalen Flughafens seit der Eröffnung des Hauptterminals 1984 und des Euro-Hubs 1991. Sie spiegelt die Veränderungen wider, die zur Integration neuer Gebäude und zum wachsenden Fluggastaufkommen wie auch infolge des Wechsels der Verwaltung von der öffentlichen Hand zu einer privatwirtschaftlichen Gesellschaft nötig wurden.

1. INTRODUCTION

Opened in 1939, Birmingham International Airport is currently the fifth largest in the United Kingdom with a passenger throughput exceeding 4.2 million passengers per annum and operating costs in excess of £43 million.

The Airport is wholly owned by the seven District Councils of the former County of West Midlands and became a Public Limited Company on 1st April 1987. The Company has a Board of Directors comprising four Executive and ten Councillor Directors. In 1991 a second terminal, Eurohub, was opened as a result of a Joint Venture between the Airport, British Airways and the private sector.

The principal objective of the Company is to operate the Airport for the benefit of the West Midlands region and to achieve a return on capital which is satisfactory to the Shareholders.

An Airport can only be successful and profitable if it makes effective and efficient use of its assets which obviously need to be maintained.

The Airport Engineering Department at Birmingham is responsible for the maintenance of the Company assets and has a budget of £5.6 million which represents some 13% of the total operating costs.

	1989/90 £'000	1990/91 £'000	1991/92 £'000	1992/93 £'000	Estimated 1993/94 £'000
Employees	1,290	1,302	1,697	1,847	1,947
Utilities	1,050	1,160	1,583	1,557	1,811
Maintenance & Repair	950	1,212	1,101	1,430	1,683
Transport	87	141	147	206	211
	3,377	3,815	4,528 *	5,040 *	5,652 *

* Includes costs for Eurohub which for the part year 1991/92 were £670,000, for 92/93 £1,156 and for 93/84 are estimated to be £1,247.

The Engineering budget for 1993/4 was set at £5.8 million which as with the previous year reflects major maintenance of plant and equipment in the Main Terminal which is now 10 years old.

Birmingham Airport operates 24 hours per day seven days a week and therefore there has to be an engineering presence at all times. Since the opening of the Main Terminal Building in 1984, the opportunity was taken to review and re-organise the structure of the Department in order to accommodate the £64 million development. Subsequent developments including the £60 million Eurohub which opened July 1991 have necessitated continual review.

The introduction of MAGLEV necessitated the recruitment of controllers and diagnostic technicians which gave the Department the opportunity of centralising works requisitions/fault reporting systems in the Control Room which also accommodates the computerised Building Management System.

2. ORGANISATION

At an operational level, responsibility for new development and maintenance is divided between a number of people:-

<u>New Development</u>

Airport Surveyor - Building and Civil Engineering Works
Airport Electrical Engineer - Electrical Works

In addition, two Contract Engineers assist with new development.

Maintenance

Senior Maintenance Manager	– overall responsibility for 80 staff
Terminals Maintenance Manager	– 33 staff
Airfield Maintenance Manager	– 21 staff
Eurohub Maintenance Manager	– 10 staff
Motor Transport Manager	– 7 staff
Engineering Support Manager	– 3 staff
Support Services	– 2 staff

All senior members of the Department have a considerable involvement with the capital new works programme (annual expenditure currently about £18 million) and other joint ventures in which the Airport is involved.

The Department, with the exception of Building Management, Maglev Control, Maglev maintenance staff and Terminals maintenance staff, is accommodated in one central complex adjacent to the Airport Fire Station.

The complex, comprising offices, workshops and stores gives the Department a central base which is linked to the central control facility enabling all engineering functions, including Maglev, to be directed from just two locations.

BIA plc, under a management agreement, provide various services including engineering to the Eurohub Terminal. However, the Eurohub Board expect the level of service they require at a cost they can afford. This led to the preparation of fully detailed and costed submission on predicted maintenance and running costs for a given level of service. The submission prepared by the Department recognised the need for a dedicated workforce to be based in the building led by a Maintenance Manager.

3. COST CONTROL AND BUDGETS

With the change of status to PLC, new accounting procedures have been introduced. Over the last two years, the Company has introduced a Business Centre approach which has involved a change to the control of maintenance budgets. The Engineering Department assists the various client departments when setting their budgets and manages them on their behalf.

Using Business Solve, monthly management accounts are produced and circulated to all budget holders. The alphanumeric Business Centre coding system used is powerful and enables the location, budget holder and type of expenditure to be identified.

eg	P2 TM 032344	
Passenger Terminal	Terminal Manager	Baggage Handling

The annual budget is determined by the Department and reflects the planned maintenance programme and the past year's performance. Contingency is provided but controlled by the Executive Management Group. The budget is submitted to the Board for final approval.

Each line manager has responsibility for managing the various client budgets but expenditure levels are imposed which ensure effective control.

With effect from 1st April 1993 a Business Centre approach to financial accountability was introduced. There are three Business Centres, namely Airfield, Customer and Property Services. Each Centre is headed by a Manager who is responsible for both income and expenditure. In addition, a sub-business centre was established for Engineering.

4. FAULT REPORTING/WORK REQUISITIONS

The fault reporting system introduced in 1989 also embraces a works requisition and hazard report system. The system has been refined and computerised so that faults can be analysed and performance and availability of plant and equipment figures produced to compare against predetermined target figures.

To enable the Department to respond to urgent problems, key tradesmen carry two-way radios and can be contacted directly from the Building Management Room.

5. COMPUTERISATION

At present within the Department, the areas of computer use are limited to:-

- Building Management System
- Energy Monitoring
- MT Work Scheduling
- Building Fault Reporting/Works Requisitions
- Planned Preventative Maintenance

The PPM system is the most recent innovation and is being introduced on a phased basis. The system went live in Eurohub in February 1992 and was extended to Main Terminal in February 1993 and introduced on the Airfield November 1993,

6. STORES AND PURCHASING

The purchasing function is controlled by the Purchasing Manager whose section is separate from the Engineering Department. However, the Engineering Stores, accounting for approximately 60% of the total items held in store and 85% of the stock value are under the control of the Engineering Support Manager who reports directly to the Senior Maintenance Manager.

Stationery, clothes, cleaning materials etc are now held with suppliers and called for directly to the end user.

Good liaison between Purchasing and the Engineering Department ensures an effective store holding without excessive costs. The stock holding of items and quantities is regularly reviewed and revised as required.

All orders for goods and services are placed through the Purchasing Department.

7. MOTOR TRANSPORT

The Section, supervised by the MT Manager is responsible for the maintenance of 100 vehicles owned by the Airport. Vehicles are generally serviced on a fixed time interval basis. Work scheduling is done using a budget software package run on a Tandon (IBM compatible). The Airport Company is now concentrating on specialist vehicle maintenance and has moved toward contract hire for 'common user' vehicles.

In addition, having resolved insurance problems, the MT section has commenced third party maintenance and also maintains the leased vehicles.

8. BUILDING SERVICES, AIRFIELD LIGHTING AND SPECIALIST EQUIPMENT

The Maintenance Managers allocate their work using a basic job sheet system. The tradesmen undertake reactive (breakdown) maintenance and minor works. Major building and pavement maintenance/alterations are undertaken by contractors.

The three groundsmen are responsible for maintaining the 750 acres of grass areas. Horticultural works are undertaken by contractors. Waste collection previously undertaken in house is now contracted out and three members of staff have been redeployed.

Airfield lighting is inspected twice daily and outages are attended to on an opportunity basis. Building Services and specialist equipment is checked daily and maintained in accordance with manufacturers recommendation and in the light of operating experience. The frequency of maintenance also takes account of hours run, seasonal opportunity and condition monitoring.

Whilst the MT Section and Building Tradesmen work alternating shifts, the Electricians and Fitters, together with the Engineering Duty Manager, provide 24 hour cover. The shift pattern is complex and provides for minimal manning at night with most of the planned work being undertaken by the early and late shifts. However, with the growth in business, management are having to review when certain PPM can be undertaken, which may result in strengthening the night shift.

9. ENERGY AND MAINTENANCE

Good maintenance, including rectification of faults, makes a major contribution to energy cost reduction. This has enabled the Airport to give a corporate commitment to energy efficiency.

The chillers are now linked to a Strainer Cycle System which uses the cooling towers to directly chill water when the ambient conditions allow. This has drastically reduced the running time of the chillers and at a cost of £100,000 has yielded savings of £70,000 per annum on electricity costs. This, coupled with an interruptible gas supply for the main boilers, has rendered the Heat Recovery System redundant and it has now been removed.

With phased removal of the Heat Recovery System, further operating improvements were made and two years ago a small scale CHP scheme was installed. The System involves two CHP units and, utilising a discounted electricity purchase option, has yielded savings of £20,000 per annum at no capital cost. Other schemes implemented include low wattage fittings on airfield lighting, metering and automatic control of internal lighting using infra–red devices.

10. SKILLS AND TRAINING

To enable the Department to cope with an ever increasing amount of new technology, particularly micro–electronics, a programme of training/re–training existing staff, including multi–skilling, has been prepared and introduced by the Engineering Support Manager.

In addition, all new project budgets include a provision for training and staff are actively encouraged to observe the installation of new equipment and be present during commissioning.

11. CONCLUSION

The maintenance strategy of the Department is the provision of cost effective services matched to the operational and commercial requirements of the Airport.

As has already been stated, Airport Engineering costs are substantial and planned maintenance is a key tactic in the maintenance strategy. Good planning, combined with effective management, maximises the potential of the maintenance resources.

Most importantly, the Department has been set up as a 'Service Company' providing a cost effective/efficient engineering service to client departments. Ultimately this will lead to direct competition with the private sector companies offering such expertise.

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Check of Roof Loading of the Westfalenhalle in Dortmund

Détermination des charges du toit de la Westfalenhalle à Dortmund

Kontrolle der Dachlasten bei der Westfalenhalle in Dortmund

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SUMMARY

In the course of strengthening the roof of the Westfalenhalle, it was necessary to check the actual roof loading. This article shows how the loads were indirectly determined by measuring the anchor forces. These were obtained by means of oscillation measurements and an exact interpretation of the frequency spectrum.

RÉSUMÉ

Au cours des travaux de renforcement du toit de la Westfalenhalle les charges effectives du toit ont dû être vérifiées. Cet article décrit la méthode selon laquelle les charges sont déterminées indirectement en mesurant les effets dynamiques sur les tirants. Ces influences sont déterminées en mesurant les oscillations et à l'aide d'une analyse du spectre de fréquences.

ZUSAMMENFASSUNG

Im Zuge der Verstärkungsarbeiten an der Dachkonstruktion der Westfalenhalle war es erforderlich, die tatsächlichen Dachlasten zu überprüfen. In diesem Beitrag wird gezeigt, wie diese Lasten indirekt durch die Bestimmung der Zugstangenkräfte festgestellt wurden. Diese Kräfte wurden durch Schwingungsmessungen und eine genaue Analyse der Frequenzspektren ermittelt.

1. DESCRIPTION OF THE HALL

The 40 year old Westfalenhalle in Dortmund has a spectator capacity of approx. 12000 - 14000. The plan dimensions of the building are approx. 110 m x 98 m in a semi-elliptic shape, with a clear height of approx. 28m.

The main structural roof elements consist of 20 riveted steel beams, which cantilever out to the centre of the roof, and are anchored at the rear by means of tie rods from the end of the arm down to the foundations (Fig. 1).

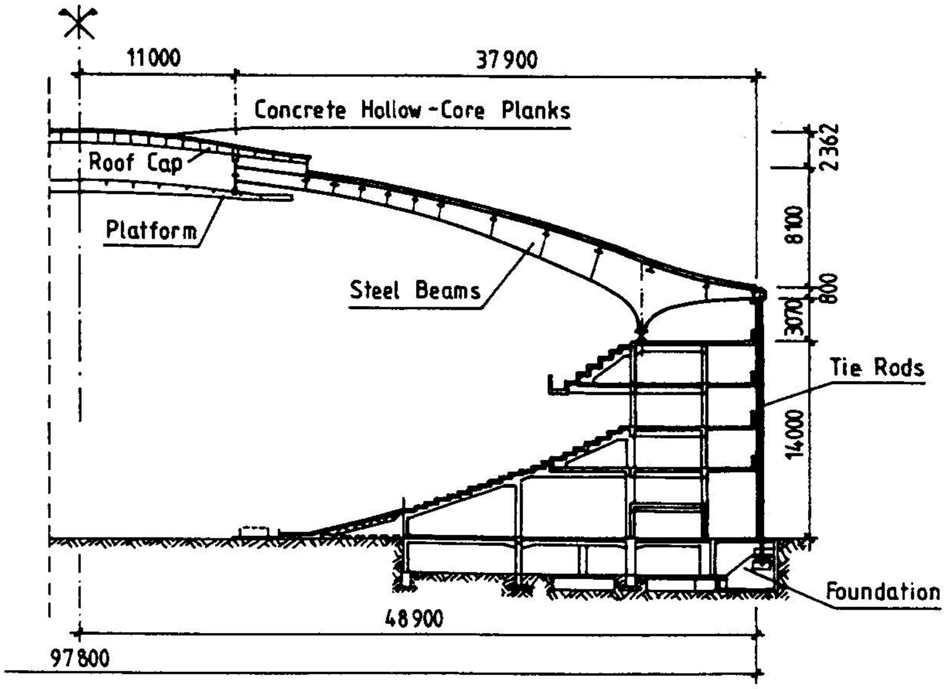

Fig. 1 Cross Section of the Westfalenhalle

The tie rods each consist of 5 separate bars with diameters varying from 55 mm at the weakest beam to 110 mm at the strongest. The clear length of the tie rods from foundation level to the beam connection is 17,34 m.

The high roof weight of approx. 1,30 kN/m^2 is due mainly to the concrete hollow-core planks used originally 40 years ago.

2. THE PROBLEMATIC NATURE OF THE ROOF LOADS

It became necessary to check the roof structure in 1992, due to the fact that beam loading during rock concerts was ever-increasing.

The total loading due to loudspeakers and stage lighting was often in the region of 300 kN. Also, due to various previous structural alterations to the roof, there was some uncertainty as to the present roof load. In order to have a safe basis for checking the structure, as well as for the preparation of any structural strengthening which might be required, the loads were indirectly checked by measuring the tie rod forces. These forces were obtained by measuring the natural frequencies of the tie rods.

3. TEST DATA AND MEASURING INSTRUMENTS

Seismic acceleration absorbers with a range of 0,01 Hz - 1200 Hz were connected to the tie rods.

The output signal from these acceleration absorbers was amplified and transferred to a monitor. In this case, the five first natural frequencies could be obtained and depicted with a great degree of accuracy.

By means of various test measurements, the influence of the natural frequencies of the absorber, the influence of mutual neighbouring rods and the influence of transverse vibrations were eliminated.

The typical simplified test result for a tie rod with a diameter of 100 mm is shown in Fig. 2. The distinct peaks of the natural frequencies 1 to 5 are clearly shown.

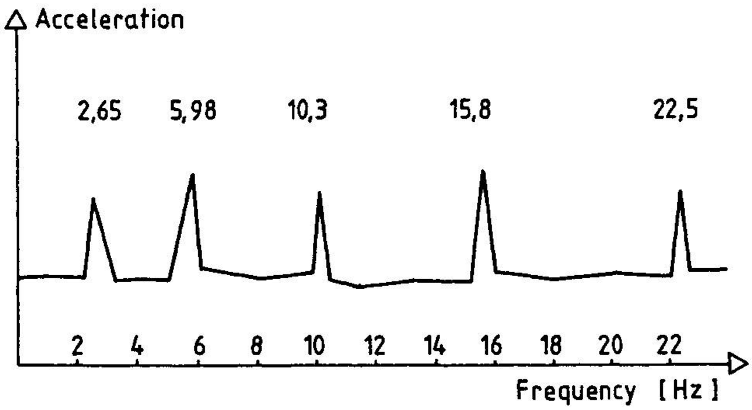

Fig. 2 Test Data (simplified) of a Tie Rod with 100 mm Diameter

First analysis of the test data led to an extremely disturbing result concerning tie rod forces, and thus roof loading. Flexible cable characteristics had been assumed for first analysis. Initially, this seemed to be appropriate for a frequency ratio of $f_2/f_1 = 2{,}26$, where f_1 = 1st natural frequency, f_2 = 2nd natural frequency. For a flexible cable this ratio is $f_2/f_1 = 2$ and for a simply supported bending member $f_2/f_1 = 4$.

However, using this simplifying first assumption, tie rod forces were obtained which were almost twice as large as expected.

The influence of the bending stiffness and of the support conditions on the actual forces is of great importance, and must be considered by further analysing the frequency spectrum, as shown in the following.

4. EVALUATION OF THE TEST DATA

4.1 Calculation of tensile forces

4.1.1 Frequency spectrum of a flexible cable

The natural frequencies f_n of a flexible cable under a tensile force can be obtained from the equation:

$$f_n = n \cdot \sqrt{\frac{Z}{0{,}408 \cdot g \cdot l^2}} \quad \text{[Hz]} \tag{1}$$

where g = weight of cable in kN/m
l = length of cable in m
Z = tensile force in kN
n = number of natural frequency

Thus for the natural frequency No. n:

$$f_n = n \cdot f_1 \tag{2}$$

If, for the tie rod of 100 mm diameter, these values are combined to a frequency spectrum, where the first natural frequency is equal to the measured frequency, a spectrum according to Table 1, line 1 is obtained, which does not conform with the test data.

	Support Condition	Frequency Spectrum Natural Frequencies No. 1 - Nr. 5	Tensile Force
1	Z EI = 0 Z Free Cable	2,65 5,3 7,95 10,6 13,25 Hz	Z = 521 kN
2	Z EI Z	2,65 5,79 9,80 14,90 21,20 Hz	Z = 487 kN
3	Z EI Z	2,65 5,88 10,08 15,42 22,08 Hz	Z = 393 kN
4	Z EI Z	2,65 5,97 10,36 15,96 22,84 Hz	Z = 290 kN
5	Test Data	2,65 5,98 10,3 15,8 22,5 Hz	Z ≈ 338 kN

Table 1 Spectrum of Natural Frequencies for various Support Conditions

The tensile force Z of a flexible cable is obtained from:

$$Z_{cable} = 0{,}408 \cdot g \cdot l^2 \cdot \frac{f_n^2}{n^2} \tag{3}$$

Using this equation, different tensile forces for each natural frequency (see Table No. 2) are obtained from the test data of Fig. 2.

		1	2	3	4
		Free Cable	Z Z	Z Z	Z Z
n	f_n [HZ]	$Z_{cable,n}$ [kN]	$Z_{2,n}$ [kN]	$Z_{3,n}$ [kN]	$Z_{4,n}$ [kN]
1	2,65	521	487	393	290
2	5,98	663	528	414	290
3	10,30	875	570	431	280
4	15,80	1158	617	449	268
5	22,50	1502	656	457	246

Table 2 Tensile forces Z [kN] for various Support Conditions

Considering these different values, it is obvious that the test data does not represent a flexible cable, because in reality there is only one actual tensile force in the tie rod. Therefore, it is essential to consider the bending stiffness and the support conditions.

4.1.2 Frequency spectrum and tensile forces with regard to bending stiffness and support conditions

The natural frequencies f_n with regard to bending stiffness, support conditions and tensile force can be determined by using the following equation (see ref. 2):

$$f_n = n \sqrt{\frac{\frac{Z}{k_1} - k_2 \cdot N_E}{0{,}408 \cdot g \cdot l^2}} \tag{4}$$

where

$$N_E = \frac{\pi^2 EI}{l^2}$$

k_1, k_2 factors, related to $\frac{Z}{N_E}$ and the support conditions (see Table 3)

	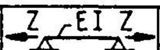		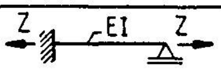					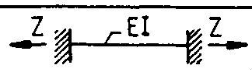				
			k_1 für Z/N_E =					k_1 für Z/N_E =				
n	k_1	k_2	10	100	1000	∞	k_2	10	100	1000	∞	k_2
1	1	1	0,897	0,949	0,979	1	2,441	0,836	0,908	0,959	1	5,140
2	1	4	0,928	0,956	0,980	1	6,406	0,873	0,918	0,961	1	9,760
3	1	9	0,947	0,962	0,981	1	12,398	0,902	0,928	0,962	1	16,673
4	1	16	0,959	0,967	0,981	1	20,382	0,922	0,936	0,963	1	25,637
5	1	25	0,965	0,970	0,981	1	30,390	0,933	0,943	0,964	1	36,598

Table 3 Factors k_1 and k_2 for various Support Conditions

For the above mentioned tie rod, the values of f_1 up to f_5 were calculated for support conditions no. 2, no. 3 and no. 4 (f_1 equal to f_1 from test data). The results are shown in Table 1, line 2, 3 and 4. These simplified frequency spectra show clearly, that the actual restraint value lies between support condition no. 3 and no. 4.

The tensile force as a function of f_n can be obtained by using the following equation where $Z_{cable,n}$ is the tensile force using equation no. 3:

$$Z_n = k_1 \left[Z_{cable,n} - k_2 \cdot N_E \right] \tag{5}$$

If all values of Z_n for support condition no. 2, 3 and 4 are determined from the test results, the results shown in Table 2 are obtained. All these values can be represented graphically (see Fig. 3).

4.2 FINAL EVALUATION OF TEST DATA

By considering Table 1 and Fig. 3 it is obvious that the actual support condition and the actual tie rod force Z lie somewhere between support condition no. 3 and no. 4.

Because there is only one single actual tensile force Z for all measured f_n in the tie rod under consideration, the force Z must be a horizontal line between support condition no. 3 and no. 4 (see Fig. 3).

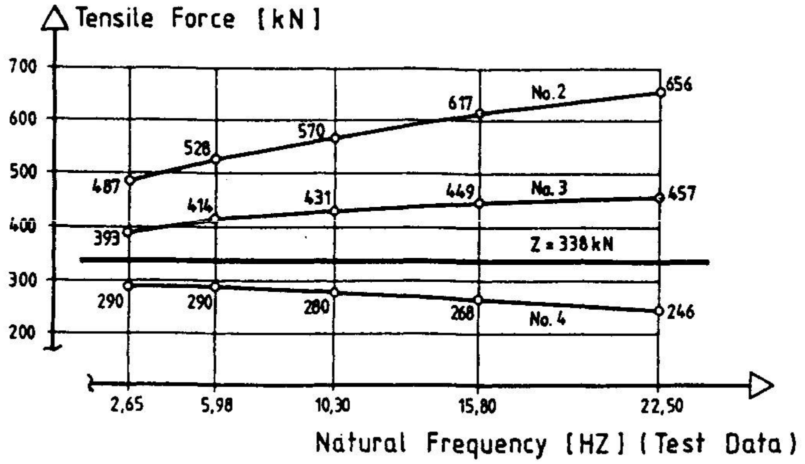

Fig. 3 Tensile Forces Z for various Support Conditions and actual Tie Rod Force

For each natural frequency, the unknown Z-force can be expressed as

$$Z_n = Z_{4,n} + \alpha \cdot (Z_{3,n} - Z_{4,n}) \tag{6}$$

The sum of the errors squared $D = \sum_{n=1}^{5} \left(Z_n - Z\right)^2$ (7) can be minimised by using

$$\frac{dD}{d\alpha} = 0 \quad \text{and} \quad \frac{dD}{dZ} = 0 \tag{8}$$

Two equations with two unknown quantities α and Z are thus obtained. The actual Z-force in the tie rod can then be assessed fairly accurately. In this case 338 kN.

Fortunately, after evaluating all the data, the measured forces were generally found to conform with the previously calculated and expected forces.

A structural concept for strengthening was then determined on this proven basis.

5. DESIGN CONCEPT FOR STRENGTHENING

In order to obtain the desired load reserves of 50 kN each per beam end, even under a full snow load of 0,75 kN/m^2, it was decided to reduce the weight of the roof cap.

The actual roofing, with a weight of 1,3 kN/m^2, was replaced by a construction weighing only 0,3 kN/m^2, by using trapezoidal steel sheeting and an insulating course. Some beams were also strengthened using stiffeners in areas where buckling was a problem.

After completion of the reinforcement work, the required load reserve of 50 kN per beam under full snow load was available, which is a total load reserve of 400 kN above hall stage, where approx. 8 beams meet.

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Structural Safety of Long-Span Building Structures

Sécurité structurale des bâtiments de grande portée

Tragwerkssicherheit weitgespannter Hochbauten

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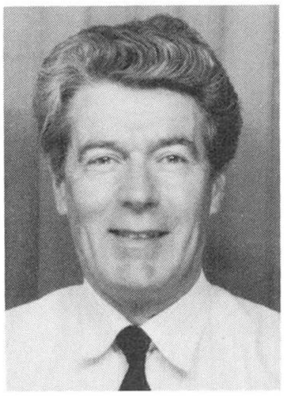

John Menzies, born 1937, is a graduate of the Univ. of Birmingham, UK. For many years he undertook research and development in structural engineering at the UK Building Research Establishment. He is now a consulting engineer and is the Secretary of the Standing Committee on Structural Safety.

SUMMARY

In accordance with its brief the Standing Committee on Structural Safety persistently questions the basis for assuming the structures are safe. Long-span structures deserve to be subject to special scrutiny and this paper discusses their distinctive sensitivities. Experience leads to the conclusion that, whilst record of safety of long-span building structures has been good, a vigilant ongoing review of concepts, techniques and materials employed in new long-span structures is necessary. General conclusions for the continuation of this good record are presented.

RÉSUMÉ

Conformément à la tâche qui lui incombe, le Comité permanent de la sécurité structurale examine avec persévérance les bases de la sécurité des structures porteuses. Les ouvrages de grande portée exigent une minutie toute spéciale et l'auteur discute de leurs points sensibles particuliers. Indépendamment d'un excellent bilan de sécurité effectué pour les bâtiments de grande portée, l'expérience montre que les nouvelles constructions requièrent un contrôle permanent des concepts, des techniques et des matériaux utilisés. Des conclusions générales y font suite, en vue de poursuivre cet excellent bilan.

ZUSAMMENFASSUNG

In Uebereinstimmung mit ihrem Auftrag hinterfragt das ständige Komitee für Tragwerkssicherheit beharrlich die Grundlagen der Annahme, Tragwerke seien sicher. Weitgespannte Bauwerke verdienen eine spezielle Sorgfalt, und der Beitrag bespricht ihre besonderen Empfindlichkeiten. Die Erfahrung lehrt, dass ungeachtet einer guten Sicherheitsbilanz weitgespannter Gebäude eine wachsame Ueberprüfung der bei Neubauten eingesetzten Konzepte, Techniken und Werkstoffe nötig ist. Es werden allgemeine Schlussfolgerungen gezogen, um diese gute Bilanz fortzusetzen.

1. INTRODUCTION

Structural safety requires there to be unused margins of stiffness and strength throughout the intended life of a structure. This guards against failure due to changes in loading or material deterioration with time.
The unused margins are influenced by:

- the design concept, assumptions and detailed design;
- the performance of the structural materials and the construction relative to the design assumptions;
- the use of the structure and its actual in-service environment compared to the design assumptions;
- the maintenance of the structure.

In many ways, the behaviour of long-span structures is more sensitive to all these influences than that of shorter spans. The sensitivities and the implications for design, construction and use are discussed in this paper in the light of references by the Standing Committee on Structural Safety to recently constructed long-span buildings in the United Kingdom[1].

2. BACKGROUND TO STRUCTURAL SAFETY

Both the traditional permissible strength margins and the probability - orientated partial factors used in modern limit state design relate only to notional margins. In design these notional margins are applied to idealised structures in simplified environments in contrast to the real structure in its actual environments.

The idealisation (model) of the structure used for analysis purposes is traditionally based on 'safe' assumptions. It usually ignores small or unreliable influences on structural behaviour which are thought generally to contribute beneficially to stiffness or strength. The result is that, from the practical experience of such structures, it is known only that, given the idealised and simplifying assumptions used in design and construction, very few collapses occur. A precise quantified measure of safety in the actual conditions cannot be derived. There is, of course, much more to the achievement of structural safety than refinement of the design model.

Whilst it is rarely possible to quantify precisely the actual ultimate strength or stiffness of a structure it is generally possible to identify hazards. These may occur in the loading or environment or from influences within construction. Such an approach reduces the risk of failure by asking the question "What if?".

In relation to structural loading and the surrounding environment, a value at the extreme of the probability model of loading can be taken. The structure can also be shielded from certain forms of loading, eg explosion, vehicle impact.

In relation to the structure itself, measures include:

- protecting the structure from aggressive actions in the environment. Coatings are used to exclude agents of deterioration, and/or materials which are resistant to deterioration may be adopted;

- aiming for a structure stronger and/or stiffer than basic margins to carry the simplified loads used in design;

- ensuring that any possible structural deterioration could be detected by routine inspection
- using forms of structure with ability to redistribute loads in the event of local failure or damage, generally obtained by providing the structure with a degree of statical indeterminacy.

In addition, measures for quality assurance in the design, construction and maintenance processes aim to give a high probability that what is done yields a safety margin as high or higher than the design intended. These measures should not be used to excuse reductions in partial factors in design but rather should provide a greater assurance that the design assumptions are sound and that the design concept has been totally fulfilled.

The achievement of structural safety is therefore not simply a matter of partial factors used in design, or even just of the design. It derives from consideration and management of all aspects mentioned above. The task is to maintain the margins against 'unsafe' conditions as a result of any of the multitude of uncertainties and hazards which may arise in the life of a structure. Structural safety has to be engineered from beginning to end.

3. SENSITIVITY OF LONG-SPAN BUILDING STRUCTURES

The safety of long-span building structures is particularly important where they are used to accommodate large numbers of people. To ensure their safety the particular sensitivities relating to their size and structural form have to be recognised.

1) With increasing span size, the stiffness and strength properties of the structural materials as opposed to the structure have a decreasing influence on structural response. The characteristics of the structure overall, ie its geometry, become predominant in determining the building's response to its loading environment. The maintenance of the intended structural geometry is affected by two factors, the relative stiffness of components and flexibility overall. Relative stiffnesses of the larger components or sub-assemblies within an overall structure have to be compatible if the structure is to behave as a unit and not be susceptible to distress caused by differential movements or undue stress concentrations. At the same time excessive flexibility of the overall structure or its major components has to be avoided since it may invalidate the geometric assumptions made in the design. Assumptions commonly made are that plane trusses have no lateral distortion and act in the same plane as the applied loads and that compression members are straight. It is more difficult to ensure that these assumptions are valid with long span structures. Secondary stresses arising through connection eccentricities and deformations may be significantly more important with long-span structures.

Ceilings and services suspended from, or rigidly attached to long-span roofs may also be affected by flexure or vibration or give rise to vibrations and lateral loads.

2) In long-span buildings the components are both more susceptible to permanent deformation due to dead loads and more flexible in their response to live load than the shorter components in other construction. The greater flexibility may require special consideration of dynamic performance and fatigue. The combination of permanent deformation and flexibility can have a direct effect on the structural geometry making these structures more sensitive to the presence of structural defects. In roofs, susceptibility to these effects is further enhanced by the increase in thermal movements with roof span (which also may be magnified due to exposure to extreme ambient temperatures) since in buildings generally, unlike bridge structures, bearings and joints are seldom specifically designed to accommodate thermal movement and overstressing

sometimes occurs. In addition, in roofs the structure is usually 'working' nearer to its design resistance than in floors because ratios of dead load to live load are higher.

3) In long-span buildings it is also more difficult to provide the structure with the ability to redistribute loads in the event of local failure or damage. In the event of a local failure in part of a long span, the whole building structure may be potentially at risk. This leads to the paramount need to ensure that such members are more than adequately strong and robust, especially if they are unable to redistribute their load to neighbouring members in the event of failure. The same considerations apply to connections.

It has to be acknowledged that there is a high price to be paid in long-span structures for any increase in self-weight. Additional robustness is difficult to achieve otherwise. Tolerable deviations assumed in design must be carefully prescribed since they affect the transfer of loads between members and the actual stresses experienced.

4. REVIEW OF EXPERIENCE

In the 1970s there were a few collapses of assembly and arena-type structures of modest spans in the UK. Most of these collapses were initiated at inadequate connections or because the structure was inherently unstable, for example, see [2-6]. The inadequacy was sometimes increased as a result of the use of weak or deteriorating materials. The origins of the structural defects associated with these experiences and more recent reports remain relevant. Similar collapses have been avoided in the UK subsequently.

Structural defects arise in one or more of the three phases of a building's history, ie design, construction and use.

4.1 Design

Structural defects have occurred through the adoption of poor design concepts sometimes without clear definition of principal load paths. A lack of competent professional engineering input in design has led to the construction of inherently unstable structures and to defects in major connections The likelihood of design not receiving the appropriate professional supervision has appeared to be greatest where modest span forms of structure, traditionally built with little engineering input, are extrapolated into larger span structures.

Many of the defects identified in the 1970s were associated with inadequate consideration of factors affecting stability and of the connections for strength, stability and movement. Omission or incorrect assessment of loading conditions was a factor in several collapses. More commonly design errors occur in the detailing of connections. Problems arise, for example, due to failure to allow for adequate access to achieve high standards of welding, to permit and ensure effective grouting and to enable thorough inspection subsequently. Explicit and critical examination in design of the essential features of the structure which ensure stability against all foreseeable hazards and close supervision and checking of the design is necessary. This should also eliminate connection defects likely to influence stability.

Other defects have been associated with the quality of the material used and its ability to retain its structural performance in the environmental conditions in the completed building. These defects led to collapse when they occurred in conjunction with stability or connection defects. Thus the incidence of collapses in new construction is likely to be reduced by making sure in design that suitable materials are specified for the environmental conditions. The design must also ensure that realistic environmental conditions are assumed and defined to those responsible for the use of the building.

In the United Kingdom moisture arising from leakage or condensation has been the most important environmental factor aggravating defects in structures. Experience shows that buildings most prone to problems associated with excessive moisture are those with flat roofs. Swimming pools create a particularly aggressive environment. Sealed roof voids allow a build up of condensation, as do large areas of single glazing or poorly insulated walls. Leaks in flat roofs may be due to a number of factors, particularly.

- (i) Lack of accuracy achieved on site making the provision of the specified plane surface to shallow falls an impractical proposition for drainage.
- (ii) Long-term deflection of components or dynamic movement due to wind action being greater than anticipated in the original design.
- (iii) Thermal movement and ageing causing waterproofing seals and membranes to crack.
- (iv) Poor maintenance of materials with limited life performance.

In general the result has been for flat roofs to 'pond' causing additional load and creating more local deflection, and for the water to seep eventually through the barriers onto the structural components. Long-span flat roofs are particularly sensitive to these factors. Leaks may then occur anywhere in the roof area. If the source of moisture is mainly due to condensation the locations found to be affected most are the bearings and joints at the ends of beams around the periphery of the building. Long-span flat roofs need to be designed to stringent requirements and with increased falls in order to reduce the risk of moisture promoting structural deterioration.

Long-span pitched roofs may experience very irregular snow loading. In particular heavy accumulations can occur at the eaves.

4.2 Construction

Some defects have arisen when suppliers of prefabricated components or sub-assemblies have acted as sub-contractors to the main contractor and the suppliers were inexperienced or were changed without the engineer's approval. These organisational arrangements have resulted in some instances in the quality of the product supplied being inferior or in the supplier being not fully aware of the context in which his product is to be used.

The likelihood that structural defects may be built into any building is influenced by the supervision employed on the site during construction, bringing into question the role of the main contractor and the client's site representative. There is no doubt that the ability and conscientiousness of the main agent, the resident engineer and local building control officer, have a profound influence on the detection and correction of defects. A proportion of defects have arisen as the result of the use of low quality materials outside the specification and poor control of workmanship on site. Omission of essential components or connections, defective assembly of connections, the use of components of incorrect size and the production of poor quality concrete on site have been factors contributing to collapses. Such defects appear to have arisen sometimes because organisational responsibilities were not clearly defined especially in sub-contracting of major sub-assemblies where divided ill-defined responsibilities for design checking and construction supervision occurred.

Factors referred to in 4.1 Design have a profound influence on construction effectiveness, eg. in welding, grouting and packing. Where these are not made as designed and where envisaged tolerances are exceeded, serious loading stresses can occur. Checking should therefore be progressive and not be left too late for adjustments to be made.

4.3 Use

Some structural defects leading to collapse of buildings with long-span roofs have arisen as a result of:

- Poor maintenance
- Substantial increases of the dead load on flat roofs caused by additional layers as a means of repair to waterproofing.
- Roofs overloaded by additional services.
- Failure to maintain the weatherproofing of flat roofs.
- Failure to maintain dry conditions in timber roofs.

The trend in the construction of arenas and stadia in recent years has been towards accommodating a wider range of facilities in increasingly large enclosures with ever greater spans. Crowd loadings and fire precautions have received far more public attention than structural adequacy. The need for engineering all aspects of the life of these large structures is clearly apparent. Innovation and extrapolation have been necessary to achieve structures worthy of the era of space age technology. There have been no reported fatalities due to the collapse of permanent arena or stadia structures, but continued vigilant inspection by knowledgeable engineers is advisable. Good access should therefore be provided by design. This will not only improve the quality of inspection but will reduce its cost.

Some striking long-span building structures have been successfully completed recently in the UK. An example is shown below in Figure 1.

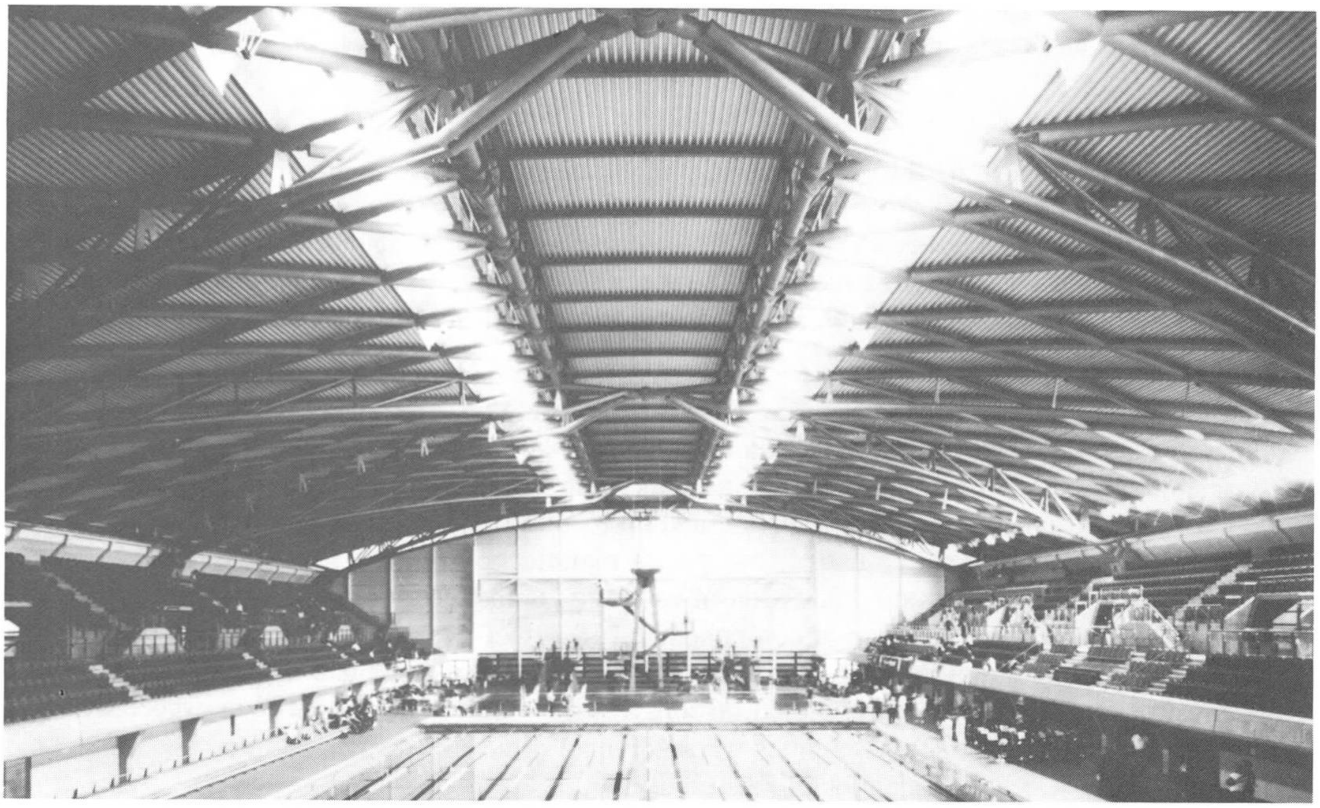

Figure 1. Swimming Pool, Ponds Forge, Sheffield (by courtesy of Ove Arup & Partners)

These structures are of substantially longer span and span/depth ratio than the examples of failure previously mentioned. They have been fully engineered to high standards throughout design and construction. The particular sensitivities of long-span structures have no doubt been recognised. It is noted that, especially in stadia structures, the structural form involves large-span trusses (120m or more) or long cantilevers (40m or more) which may be subject to excitation by wind action.

Roof structures may comprise statically determinate primary elements with limited redundancy to redistribute loading in the event of single element failure. Such long-span structures appear to be at the present limits of size for beam-and-column forms of structure. Where longer-span buildings are required, alternative forms possibly incorporating major tension cables, eg. the Georgia Dome in Atlanta, may be used.

General conclusions relating to long-span building structures are discussed below.

5. DESIGN

In view of the greater sensitivity of long-span building structures, the generalised criteria from codes prepared for normal span structures may not be sufficiently strict or comprehensive. It is necessary to work from first principles and to use realistic design assessments of environmental conditions. The structural material specified must be durable or protected where the environment is likely to be aggressive, eg in swimming pool buildings.

There are several ways of making design criteria more stringent. It is usual in the initial design of structures to assume connections are fully effective. Design analysis often does not take account of possible failure of connections but rather concentrates on the anticipated structural form. No checks are made on the influence of realistic connection performance on the behaviour of the whole structure. Such checks on the sensitivity of long-span structures to loss of connection effectiveness and the use of larger margins of safety at critical connections are ways of introducing appropriately strict design criteria.

The requirements for inspection and examination of a long-span building structure have implications for the design. Clearly these operations are facilitated if the structure is visible and easily accessible. Likewise structural safety is more likely to be achieved if the form and type of structure is one which would give early visual signs of local distress or defects should unforeseen damage or deterioration occur. Essentially it is an advantage for as much of the structure as possible to be easily visible and not hidden. These considerations are often compatible in modern long-span building structures with the design for the architectural expression of the structural form.

There should be competent independent checking of the structural design. The checks need to make sure, in particular, that possible hazards and modes of failure are identified and that either the design safeguards the structure against them or that the structure is protected from the hazard.

6. CONSTRUCTION

In view of the greater sensitivity to structural defects of buildings with long-span structures, it is especially important that construction is to a high standard and all defects avoided at that stage. Particular attention should be paid to those parts of the structure which will be hidden on completion to ensure that they are as the design intended.

The closest possible supervision of construction is needed. This should give particular attention to any elements critical to stability and safety. The client's engineer or an independent engineer inspecting during construction should be able to provide certification on completion.

7. SUPERVISION AND INSPECTION

Professional engineering input to inspection is needed even where spans are modest. Appropriate professional engineering supervision and inspection is therefore especially important in design of long spans.

8. USE AND MAINTENANCE

Regular maintenance inspection and structural examination are needed to assure future safety. A permanent and comprehensive record should be kept and reviewed by a suitably qualified engineer.

A distinction is made here between normal maintenance inspection and structural examination. Normal maintenance inspection is assumed to be visual only from points of easy access. Structural examination is a higher level of inspection undertaken by a professional engineer making a full structural appraisal. Such examinations should be regularly scheduled during the life of a long-span building structure.

Structural examination is normally visual in the first instance and includes critical examination of the design. It also involves closer inspection of the structure by access to roof and ceiling spaces or, if necessary, exposure of critical components to allow visual or other examination.

It is particularly important for buildings with long-span roofs to be maintained so that the environmental conditions and the dead load are not allowed to depart substantially from the assumptions of the design.

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