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Front cover: Roof at Schlumberger Cambridge Research new laboratories
Back cover: London Zoo canopy (Photos: Ove Arup & Partners)

Strengthening techniques

Poul Beckmann

This paper was given at the Symposium on Building Appraisal, Maintenance and Preservation held at the Department of Architecture and Building, University of Bath, 10-12 July 1985.

The saying that the end directs and sanctifies the means, applies very much to the strengthening of existing structures: the techniques we use must depend on what we want to achieve and what we want to achieve will depend very much on the type of building we are dealing with.

There are buildings which are of such outstanding merits of antiquity, architecture and artistry that they qualify for the definition of 'Historic Monument'. For these, Article 5 of the 'Venice Charter'[1] states: 'The conservation of monuments is always facilitated by making use of them for some socially useful purpose. Such use is therefore desirable but it must not change the layout or decoration of the building. It is within these limits only that modifications demanded by a change of function should be envisaged and may be permitted.' In other words: conservation is the first priority (and this is usually the popular consensus for such buildings) and if this means limitations on the possible use, this must be accepted.

Counsel of perfection

On such structures one should do as little as possible commensurate with the need for adequate safety. New work should be distinguishable from, but still in keeping with, the old and any such intervention should ideally be reversible so that if better methods are developed in 50 years time, our crude efforts can be removed without harm. Such, at least, is the counsel of perfection from art historians and conservation specialists.

Even for buildings of this nature there are, however, circumstances when slightly more drastic surgery may be necessary, possibly not to prevent imminent collapse, but to halt or slow down a gradual, progressive, reduction of safety margin.

In such a situation, one must acknowledge the natural loadbearing mechanism of the structure and restore or augment its capacity, if necessary by using modern technology. One must NOT violate the original structural principle, even using traditional materials by, for instance, building brick piers under the crowns of stone vaults! To use a parallel from the world of surgery: you don't today amputate the leg and give the man a crutch, if you can give him a replacement hip joint. It should go without saying that when assessing the need for, and the degree of, such strengthening, the precepts in Chapter 3 of the Institution of Structural Engineers' report, *Appraisal of existing structures*[2], should be followed, i.e. those constituents of the factors of safety which, in conventional design, cover uncertainties in the assessment of loadings and materials' strengths, can be reduced, subject to these uncertainties being limited by measurements and testing. (The fact that a building has stood up for 100 years is a load test in itself.)

A fair proportion of structural strengthening is associated with buildings which may be worth preserving, but only if a profitable use can be made of them. This means that making the structure fit for the intended, and often changed, use takes priority and some limitation on conservation must be accepted. This kind of refurbishment often involves some gutting and stuffing and, in keeping with this, a greater freedom in the choice of techniques can be allowed. Nevertheless, one should aim to observe the injunctions which apply to historic monuments, particularly in respect of working with the grain of the structure, as economic restraints will in any case encourage maximum utilization of the existing load-carrying capacity. Moreover, the owner will usually only be looking at a further term of use of about 50-60 years, (against the 500 years usually demanded by the custodians of cathedrals and the like) so that extreme measures to ensure durability are not required, except where the original construction has a potential self-destruct element such as a clinker concrete filler-joist floor in a damp environment.

Finally, there are structures where something has gone wrong or where weathering or corrosion has affected it to the extent that something has to be done. For these, almost anything goes: the object of the exercise is to make the building safe and serviceable, with the least outlay of money, time and disruption of use. In such circumstances, one can sometimes be justified in ignoring the residual capacity of part of the original structure and provide new load paths through new structural members. This does not necessarily make the task simpler: many, if not most, of these buildings are of fairly recent construction; their troubles may have fired the imagination of journalists and union safety spokesmen alike and whereas a 5mm crack in a building 250 years old tends to be accepted with equanimity, a 0.5mm crack in a 25-year-old building is not.

Strengthening materials

Whatever the age or prestige of the building, the techniques to be used for strengthening a structure must take account of its material and its form of construction. The material used for the strengthening may however, in some circumstances, be different from that of the original structure.

Professor Heyman has stated that 'Masonry is supposed to crack and it will do so' and goes on to imply that, given appropriate geometry and sound foundations, its cracks do not matter as far as structural safety goes[3]. True, but once a crack has formed, it tends to open and close with seasonal variations in temperature and moisture and each time a crack opens, mortar debris falls into it and gets trapped as it tries to close, thus gradually wedging the crack wider and wider. If the face of the masonry or, worse, its plastered surface is open to view, the cracks are likely to cause concern and cosmetic repairs, to keep out the weather, tend to be distressingly short-lived.

In such circumstances, it may be desirable to provide a tensile restraint against further opening of the cracks by means of tie rods. These can be external to the masonry as in past strapping of church towers, or they can be placed in holes drilled longitudinally within the thickness of the walls as in the central tower of York Minster. Alternatively, one can form horizontal rebates in the brickwork or masonry and then mortar in reinforcement bars to provide some horizontal tensile strength.

This technique also seems appropriate where some provision of tensile restraint is required to strengthen existing masonry structures against earthquakes.

Stabilizing facade walls

A variation on this theme is used when refurbishing Georgian and Victorian terrace houses which have been built without proper bond between the facade wall and the crosswalls. It is not unusual to find structural gaps of 20-30mm at these junctions and, to stabilize the facades and ensure that the filling of those gaps remains effective, one can form rebates half a brick (100mm) deep and one course high at approximately 1.0m vertical spacing, and mortar in bars bent at a right angle. Alternatively, one can precast reinforced concrete 'elbows' and bed them into the rebates with a mortar of similar properties to the existing. This has the advantage of allowing frictionally restrained slip along the joints as opposed to the mortared-in bars which, in case of a strong enough movement, may tend to tear out large chunks of brickwork, bordered by diagonal cracks which may offer only poor resistance to further movement.

One of the major bugbears of masonry is inherent in the common form of construction consisting of two skins of dressed stone with a core filling of rubble and mortar. Unless there are fairly close-spaced bonding stones going from one face to the other, such walls are very vulnerable to frost wedging, if water is allowed to enter the core because of bad roof maintenance, etc. There have been cases when the top of the core cavity has been found empty and the bonding stones have been pulled in from the outside face of the wall, whilst the lower part of the wall was bulging from the excess material which had fallen into the gaps in the lower core, left after frost expansion.

For this condition two things need to be done: any cavities in which water might collect and freeze must be filled to prevent further deterioration and the two skins of the wall must be tied together to enable the wall to function as one entity. The first, the filling of cavities, can be achieved by grouting. The second will however require either the insertion and fixing of numerous bonding stones or the provision of metal ties, fixed at each end in the skins and bridging the core or cavity. Grouting, on its own, is most unlikely to glue the two skins together.

Anchorage of ties

The anchorage of the ties in the skins can be by expanding shields or by resin anchors, each of which has advantages and disadvantages, or it can be effected by using ribbed reinforcing bars (of stainless steel or galvanized) and grouting them in at the same time as filling the voids.

Masonry spires of certain kinds of stone can sometimes be worn so thin by weathering, that their wind stability becomes endangered. If the supporting tower can carry a slight increase in vertical load, the stability problem can be overcome by constructing a bonded masonry lining, which will add both stabilizing dead load and strength to the existing shell of the spire. This technique was used to stabilize the 42m high, 6.5m wide spire to Holy Trinity Church, Coventry, where the sandstone masonry shell on the lower parts had been worn down from about 0.15m thickness to 0.12 with isolated spots as thin as 0.08m. The brickwork lining was 0.23m thick around the lower window, 0.11m on the solid faces of the octagon for the lower 10m and 0.08m on the remaining 9m height of the lining[4].

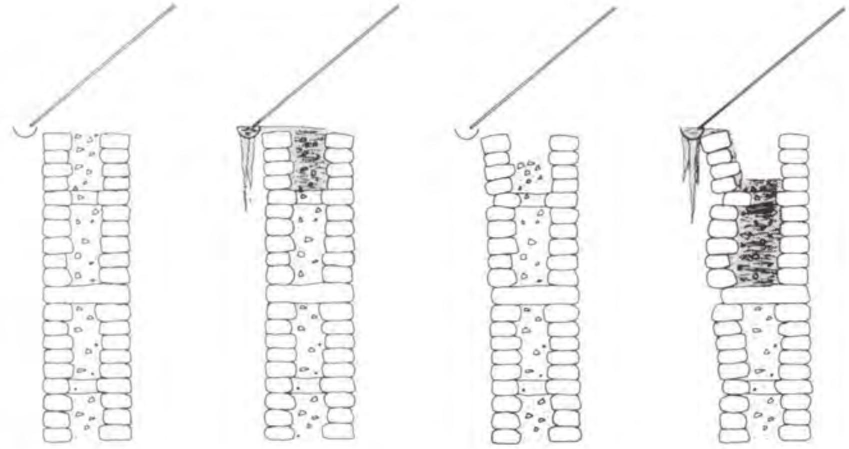

Fig. 1
Deterioration of ashlar and rubble wall caused by frost expansion in rubble fill saturated from leaking gutter

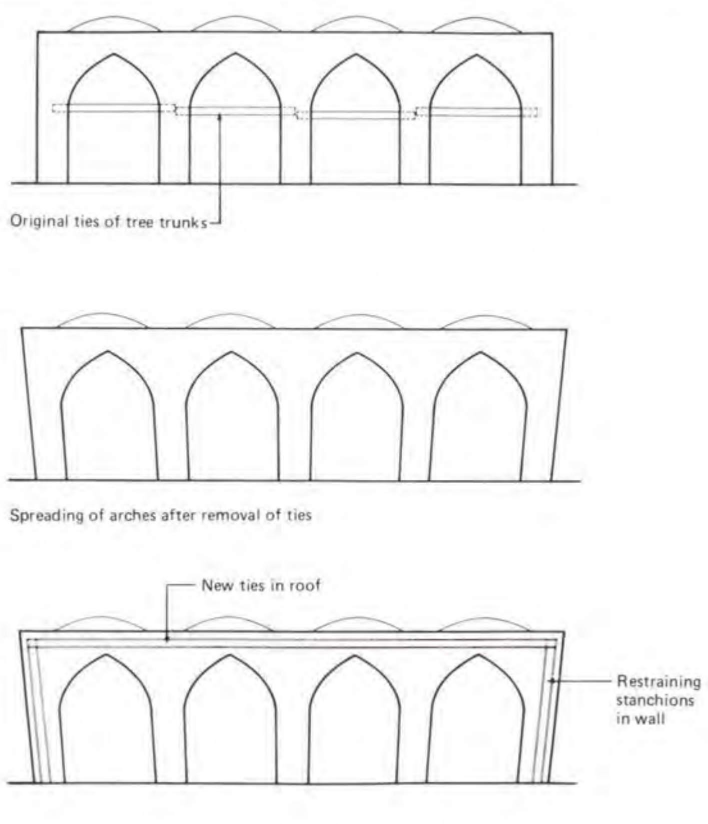

Fig. 2
Koran School in Rezaiyeh: arch spreading and remedial work

A very special form of masonry is the mediaeval window tracery. The resistance against wind loads of such windows has usually little to do with bending and must be created by arching within the depth of the structure with the window surround acting as abutments. It happens however that such abutments give to the extent that the arch action is put in jeopardy. For modest sized windows, it may be adequate to fill the opening joints. On a very large stained glass window, which may have a double mullion system, the original structural mechanism of wind resistance may be difficult to understand and if large permanent deformations are observed, one has the choice between rebuilding (which is likely to damage the glass during its removal) and an external bracing system of catenary cables with struts or suspender straps. Such a system has been applied to the great East Window of York Minster and is described by Dowrick and Beckmann[5].

And, to revert to the masonry arches and vaults: There are instances when the original builders have misjudged the ground conditions or have got their geometry wrong, or later generations have interfered with the structure. This usually results in slow spreading of the arches or vaults with correspondingly increasing tilt of the external buttresses and walls.

The traditional remedy is to provide further buttressing. No foundation will however carry any load without settlement, so the new buttresses will not have any beneficial effect until the soil under their footings has been compressed to take the load, by which time the soil under the original footings may have completed its consolidation and

Fig. 3
Koran School in Rezaiyeh: view from adjacent roof

Fig. 4
Koran School: new tie in the roof

Fig. 5
Koran School: restraining stanchion in the wall

(Photos: Poul Beckmann)

The restoration of the school was designed by the National Organization for Preservation of Ancient Monuments in Iran

the movement ceased. The consequences of this are occasionally observed in buttresses which are hanging from the original structure and are slowly pulling away from it.

What is really wanted is a tie, preventing the springings of the arches from moving apart. Physical ties at springing level are however architecturally undesirable and it is only on rare occasions that one can transfer the force to the springings from a tie located at an invisible, or visually unimportant level; but it can sometimes be done.

Turning to our other traditional structural material, timber, the most usual cause of the need for strengthening is decay or insect attack.

Rot usually develops first in the ends of beams in external walls or in wallplates and feet of roof trusses and rafters. The principles of most strengthenings in this situation are (a) to remove the affected part completely, in order to prevent spreading of dry rot, and (b) to provide a substitute structural member or part of member. The surgical parallel: amputation and prosthesis. The substitute member can take many forms: a new length of timber, pieced in, a pair of steel channels, bolted on, glass fibre reinforced resin casting, etc.

Beetle attack tends to be general rather than localized, so it is usually a case of leaving alone for a further period, if a structural appraisal so permits, or total replacement.

When a deficiency of load-carrying capacity dictates that strengthening of a timber structure is required, the possibilities are many.

Floor joists, hidden by ceilings, can be doubled up; bigger beams, which today cannot easily be replicated, can have steel plates bolted on their sides or steel beams can be inserted between them. It must however be remembered that naked steel is likely to have a much shorter period of fire resistance than the original timber structure. This problem may, paradoxically, be overcome by encasing the steel with timber which, in a thickness of 40mm, may provide up to 1 hour's fire resistance to the steel.

Roof trusses are sometimes strengthened by the provision of additional members.

An example is provided by the roof to the South Transept of York Minster which was destroyed by the fire in July 1984: as originally built in 1755 (after an earlier fire!) it had collar-beam trusses in order to clear the ridge of the vault. Their inadequate stiffness caused them to exert thrusts on to the unbuttressed clerestory walls, causing them to lean outwards. Street strengthened and stiffened the trusses in 1874 by bolting on planks which effectively changed the structure into scissors trusses.

If the timber structure is visible, strengthening does however become a more tricky exercise if it is not to be architecturally damaging.

For floors with exposed beams it has been suggested that some tensile deficiency may be made good by a steel strap screwed to the underside of each beam whilst a compressive flange is formed by closing up the floor boards and screw-fixing them to the beams. In view of the relatively poor compressive strength and modulus at right angles to the grain (particularly for softwoods) it may be better to replace the floorboards by hard plywood. When such strengthening merely provides a margin of safety for live load of people, the low fire resistance of the steel need not be a problem, as the building will have been evacuated before any significant weakening has occurred. As far as appearance is concerned, if the original timber is dark, a steel strap, painted black, will not be visually too intrusive.

The steel strap idea has been adopted to strengthen the tension booms of a series of hardwood roof trusses over a gymnasium when it was found that the split-ring connected splices had failed due to excessive slope of grain. In this case the steel straps provided minimum visual disturbance to the original design as opposed to the alternative of interspersed castellated steel beams as relieving structure. Being a roof, there were no requirements for fire resistance.

When considering repair or strengthening of structures of ferrous metals, one must distinguish between cast iron, wrought iron and steel.

Cast iron, such as may be found in older structures, is brittle, hence factors of safety, particularly against bending failure, have to be much higher than for wrought iron and steel. Because the melting point of cast iron is lower than that of iron oxides it cannot be welded by normal techniques.

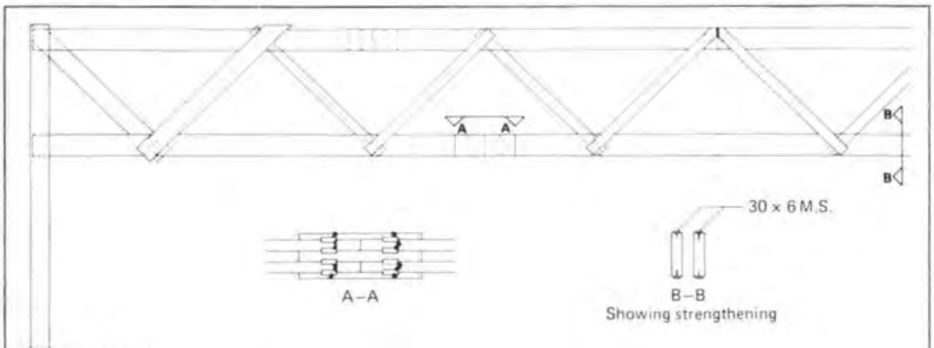

Fig. 6
Strengthening of failed tension chord of timber truss

Cracks may be repaired by bronze or brass stitches formed by machining dumb-bell shaped slots into which the alloy is cast, but this requires removal to a workshop. Drilling holes in situ for bolt-on strengthening plates, etc., can be difficult because of the hard skin which is often found on cast iron, so there appears in general to be very limited scope for in situ strengthening of cast iron structural members.

Although wrought iron is ductile, its mechanical properties are far more variable than those of steel. Building inspectors are therefore likely to impose conservative factors of safety, albeit not as onerous as those for cast iron. The laminar nature of the material means that it is unsuitable for welding, but bolting should be less of a problem than for cast iron. Nevertheless, the drilling required for bolting on an entire flange plate would be a daunting task, but it may well be possible to attach such brackets and struts as would enable a bowstring tie to be used, if it were otherwise appropriate.

Of all structural materials, steel, and particularly mild steel, probably offers the widest scope for modifications strengthening and repairs, due to the ease with which it can be cut and welded. Strengthening a simple rolled section by welding on extra plates is generally straightforward. Compound beams are more tricky because of the rivet heads, but provided that one is not concerned about painting the resulting gap, extra flange plates can be attached by welded-on distance pieces. If the stiffness of the floor beam in a frame is grossly deficient and the stanchions are alright, one can, headroom permitting, double up the beam by welding another one underneath; this may seem extravagant in the use of steel but may be worthwhile in terms of minimum disruption.

If the beam is partly embedded in a concrete floor and is of the oldfashioned compound type, it may even be possible to consider the rivet heads as shear connectors and show that if composite action of concrete and steel is allowed for, the beam may not need strengthening at all. (This approach can of course be equally applied to wrought iron.)

And so to reinforced concrete, the wonder material of Modern Architecture, the *bête noire* of the urban sociological journalist and the happy hunting ground of 'experts' with detailed knowledge of codes of practice and little design experience.

Where the defects are mainly the manifestations of inadequate durability of the original construction, there are a number of techniques available which will restore defective cover and protect the reinforcement against further corrosion. These are adequately described and discussed in the recent Concrete Society Technical Report, No. 26: *Repair of concrete damaged by reinforcement corrosion.*

If the safety of a structure has been seriously reduced by deficiencies in the original design and/or construction or if deterioration cannot practically be arrested, more drastic treatment becomes necessary.

Leaving aside the obvious, and sometimes unavoidable, measures of demolition and reconstruction, there are, in principle, two possibilities: to improve the strength of the existing members or to introduce new structural elements to carry some or all of the load and thus relieve the defective parts.

Improvement of the strength of existing members can be achieved in many ways:

Slabs may be strengthened by replacing the screed by a bonded, reinforced, structural topping which can be floated to give a surface that will accept normal floor finishes. Scabbling of the original slab will usually be required and it may be necessary to introduce drilled-in shear connectors in areas of high interface shear or at edges of the strengthening, where peeling stresses might otherwise be worrying.

If a shortfall or displacement of top steel is discovered before the slab is screeded it will usually be more convenient to cut grooves into which additional reinforcement can be bonded by dry-packing with a polymer-modified cement mortar; (grouts will shrink, leaving doubts about the efficacy of the bond; epoxy resins and mortars require very careful mixing and surface preparation which is difficult to ensure).

The strengthening of beams proves rather more difficult: extra top steel may be accommodated in the slab alongside the original cage in a similar way to that described above. (If the shortfall is slight, it may however be possible to show that there is sufficient distribution steel in the slab which can be taken into account to provide the required cross-sectional area.)

Experimental work is being carried out at Sheffield University, amongst other places, to evaluate the performance of steel plates glued to the faces of reinforced concrete beams with epoxy resins. When glued to the sides of the beam such plates can provide extra shear strength as well as bending reinforcement. The reliability of in situ gluing under practical conditions remains to be proven and protection must be provided to prevent glue-line failure during a fire.

If adequate anchorages can be accommodated at the ends of the beam span, it may be possible to arrange a prestressing cable either side of the beam.

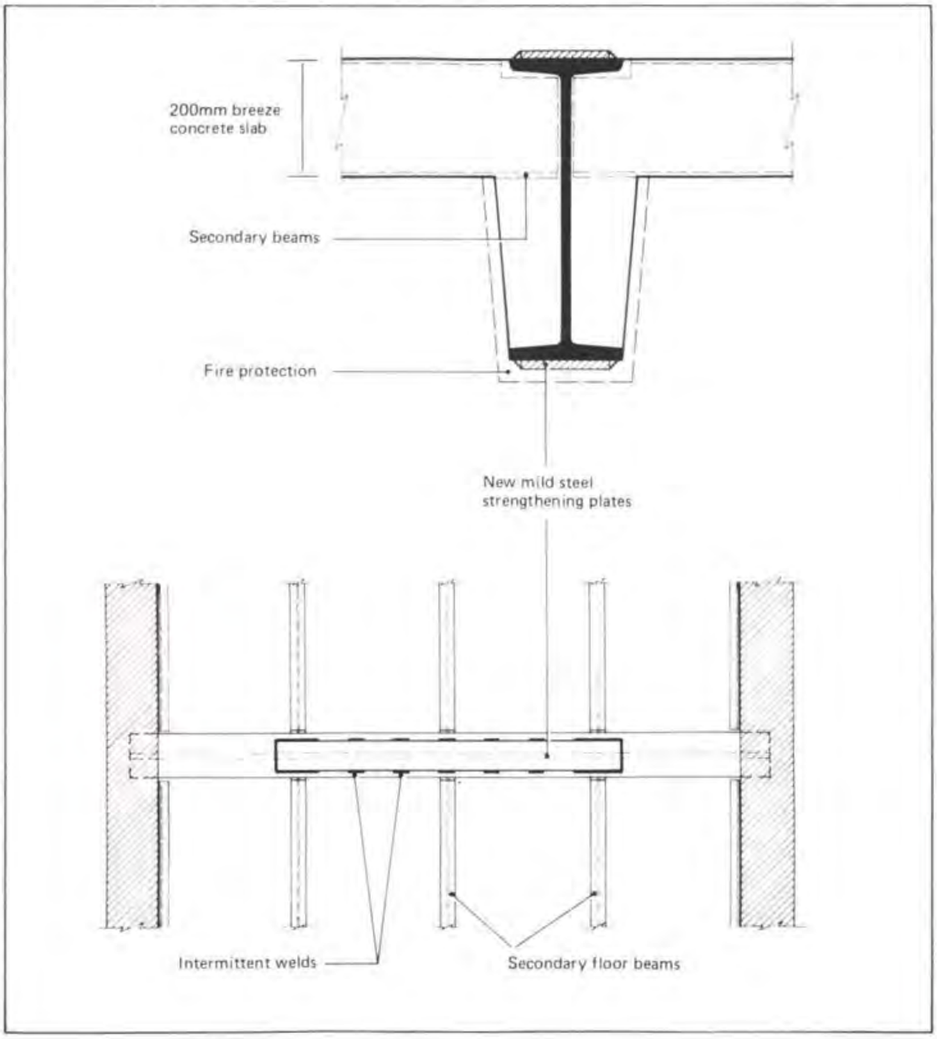

Fig. 7
Strengthening of old steel beams on Savoy Court

Fig. 8
Welding of strengthening plates to floor beams on Savoy Court (Photo: Frank Pyle)

Fig. 9
Office block with loadbearing facade elements of precast concrete
(Photo: Poul Beckmann)

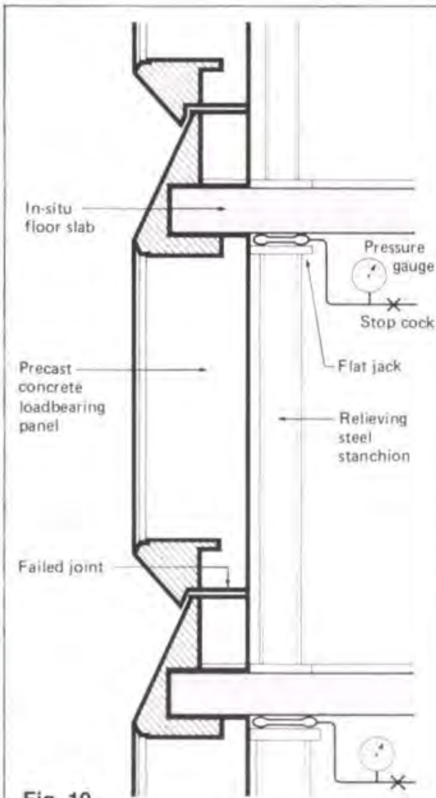

Fig. 10
Section showing precast concrete facade elements and remedial work

Fig. 11
Close-up of flat-jack arrangement at top of relieving stanchion
(Photo: Poul Beckmann)

The cables would be deflected under heavy dowel bars placed in holes, core-drilled through the beam web. The cables would have to be protected against corrosion and fire, but subject to the slab, forming the compression flange, having an adequate reserve of strength, this technique provides extra shear capacity as well as improved bending resistance.

Additional shear strength of reinforced concrete beams can sometimes be provided by external stirrups. In one instance a baling machine is said to have been used to wrap half-inch steel straps round rectangular beams, which had cracked in shear from having inadequate (but code conforming!) stirrups. For T-beams, pairs of holes, either side of the web, will need to be drilled to allow the threaded ends of U-bars to pass through so that they can be anchored and tightened by means of nuts bearing on top of the slab.

A shared problem

With the exception of the prestressing cables and the screwed stirrups, all the techniques outlined above share to a greater or lesser extent the problem that the strengthening does not begin to take any load until (further) deflection takes place. If the extra reinforcement is mortared in before the formwork is removed, it will behave as if it had always been part of the initial construction.

In the other instances, the original reinforcement will be stressed above the usual limits by that part of the load which is shared between the original and the extra reinforcement. This does not matter as long as the original reinforcement does not exceed its yield stress under working loads and deflections and cracks remain within acceptable limits. Otherwise it may be necessary not only to prop, but to actively jack up the dead load deflection to ensure that old and new reinforcement work elastically together and that the new steel assists as much as possible in reducing deflections.

The same difficulty of making the strengthening immediately effective is encountered when strengthening columns by casting (or guniting) a casing or sleeve around them, but it is aggravated in this case by the shrinkage of the new concrete.

(For round columns and for square columns with relatively thick casings it could however be argued that the fact that the casing does not carry any load and, indeed, because of its shrinkage, helps to overstress the original column, does not matter because the casing will shrink on to the original column and, if suitably reinforced, will put the original concrete into tri-axial compression and thus enable it to withstand much higher stresses.)

The initial non-effectiveness of added concrete is much less significant when the strengthening is intended to increase the bending resistance rather than the axial load capacity of a column, e.g. when improving the resistance against vehicle impact of freestanding columns close to a road.

If, in an otherwise sound structure, certain members have become damaged to the extent that their future load-carrying capacity cannot be relied upon and if at the same time it is very difficult to remove or reconstruct the affected members, it is worth considering the creation of alternative loadpaths through new structural members.

If such new elements have to be introduced in an existing building, it is often less troublesome if they are of structural steel, so that their erection becomes largely a dry operation. Steelwork has the additional advantage, for this application, of not suffering from shrinkage and creep, so that one only needs to deal with elastic deformations, by jacking or wedging, to ensure that the by-pass structure becomes immediately effective.

An example of such a by-pass structure was provided by an office block with 13 storeys of precast concrete loadbearing facade panels supported off a transfer structure at 1st floor level. The horizontal joints between the panels had been badly executed and, as a result, the loadbearing mullion parts of the panels on the lower floors began to split badly when construction had reached the 12th floor. As reinforcement loops from the panels interlaced with the bars in the in situ floor slabs, panel replacement was considered impractical. Structural steel stanchions were therefore inserted immediately behind the mullions and to avoid further, delayed, cracking of the concrete as the steel stanchions were gradually taking up their load, they were pre-loaded by means of flatjacks. As the floor slabs were incapable of sustaining the punching shear from the stanchions carrying 12 storeys, the stanchions on any one vertical line had to be pre-loaded simultaneously, and to avoid excessive distortion and hence cracking of the facade panels as they were relieved of their load, the stanchions adjacent to those being fully pre-loaded were partially pre-loaded at the same time!

Strengthening techniques can thus be seen to range from simple carpentry to complicated application of latter-day technology and materials. The choice must depend on the nature of the original construction, and the aim should always be to retain as much of the function of the original structure as is compatible with the safety and serviceability of the repaired structure.

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The engineered use of coated fabrics in long span roofs

Brian Forster

Woven fabrics have been used for centuries in the manufacture of tents for both domestic and military shelter but only during this century have fabrics been deliberately engineered into machines and structures to be capable of sustaining variable wind, snow and temperature loads whilst maintaining predictable and appropriate degrees of stability.

Some obvious examples are sails, tyres, balloons, airships, aeroplanes and aerostats, inflated boats and bridges, fascines, dams, lagoons, and storage vessels.

More recently, coated fabrics have been engineered into roofs for buildings in the form of air-supported roofs, and prestressed membranes. There is a strong temptation to call the latter simply tents – they use tent-like materials and look superficially like tents but differ from traditional ones in that they have deliberately arranged surface curvatures and are deliberately put into a state of tension.

Both types' performance is dependent on the co-existence of surface curvature and prestress. Curvature is required as the basic means for transmitting laterally-applied loads. The in-plane membrane forces developed by the externally applied lateral loads are inversely proportional to the radii of curvature. Prestress is required to maintain a residual tension under the application of non-uniform applied loads and to prevent excitation.

Prestressed membranes

Prestressed membranes are doubly-curved saddle surfaces and at every point within the surface there are curvatures opposing one another, see Figs. 1 and 2. Prestressing is achieved by mechanically stressing one set of membrane yarns against another. The more curved the surface, the more effective the prestress as a means of providing surface stiffness. Large variations in curvature across the surface can lead to substantially different properties, e.g. a soft area or a stiff area, in different parts of the surface.

Surfaces which are easy to prestress are also surfaces which have relatively uniform curvature throughout.

Large-span structures are often created by reinforcing the membrane surface with systems of ridge cables which in turn are supported by masts. Another method is the use of bending stiff arches, portal frames, and lattice shells, see Figs. 3 and 4.

One of the features to be noted with tensioned roofs is their response to dynamic oscillation, due to either earthquakes or wind buffeting. Usually their low mass and their high frequency put them outside the range of most natural phenomena, and they do not present problems.

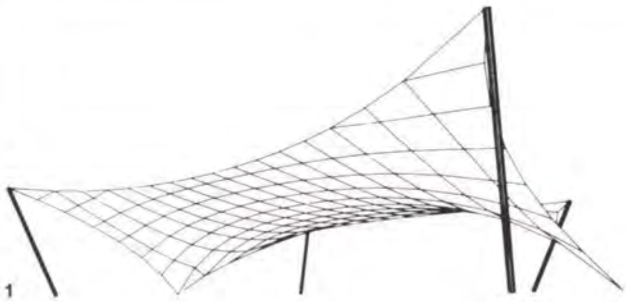
Air-supported structures

Air-supported structures are convex or cylindrical membranes which are prestressed through an internal air pressure, see Fig. 5. In this situation the crucial supporting element is the internal air itself, and the distortion of the air mass and structure under application of loads is clearly large and dependent upon the particular load distribution. Numerical modelling is now capable of describing the gross surface of deformations which these structures undergo during extreme wind and snow conditions, and also during inflation.

The form of air structure is not as restricted as one might think from looking at those normally available. Modelling techniques show that, by introduction of stiffer elements in the surface, free and unusual forms can be achieved[1]. Although the ideal shape is spherical, considerable variation from the sphere can be tolerated as the stress levels are normally not high. This means that they are not sensitive to fabrication error or material quality and consequently the largest structures tend to be air-supported. A critical factor in the decisions on the shape is the avoidance of instability under wind buffeting.

The natural frequency often falls within the critical range and the method generally adopted in practice is to change the internal air pressure as movement of the surface starts, thus changing the natural frequency and hence the response to buffeting.

Changing the air pressure can be done via an automatic control system related to wind speed and direction and calibrated for each particular site. Air-supported roofs must fail-safe as far as people inside the building are concerned. This means that in the event of loss of pressure the membrane can collapse but must remain suspended clear of the human occupancy zone. Stadium roofs achieve this by virtue of their attachment to a perimeter ring beam elevated above raked seating and the field. In the inverted/deflated condition such roofs need to be equipped with flap valves at appropriate positions. This is to avoid rainwater ponding and collapse. The possibility of rainwater being deposited inside, albeit infrequently, limits the type of use to which such a building can be put.



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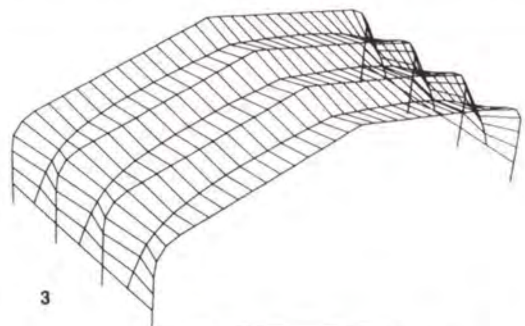

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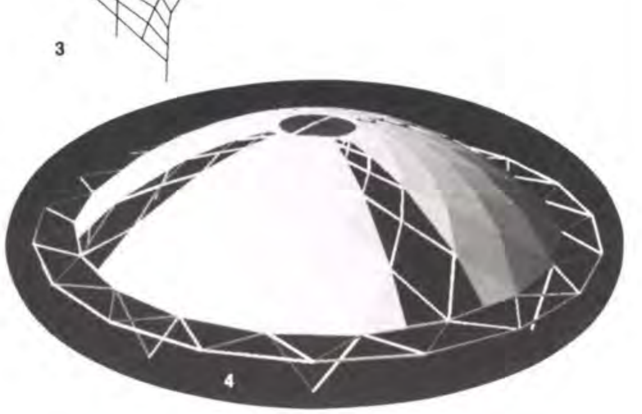

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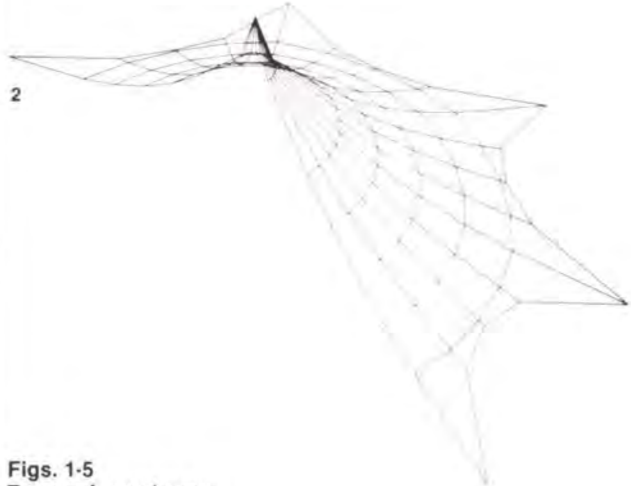

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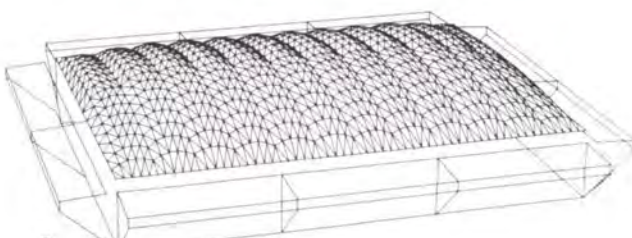

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Figs. 1-5
Types of membrane used in long span roofs

Some Arup projects with fabric roofs

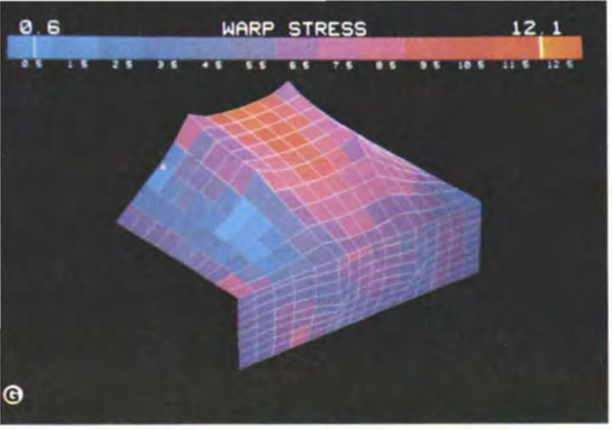
A-C: Mont Rouge, Paris
Covered pedestrian route
between landscaped buildings

D-E: Clifton Nurseries, Covent Garden

F: London Zoo
Canopy over a stage
and terraced seating

G-I: Schlumberger Cambridge Research
New laboratories

J: Yulara International Tourist Resort,
Australia. Membrane sails



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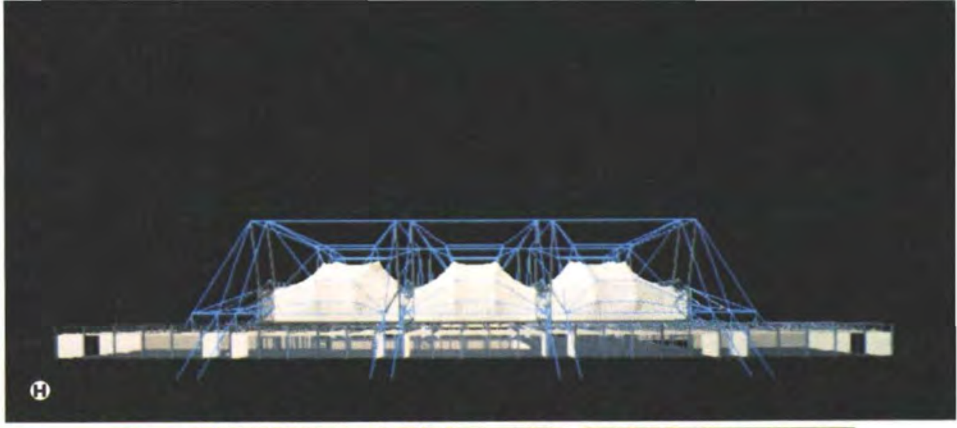
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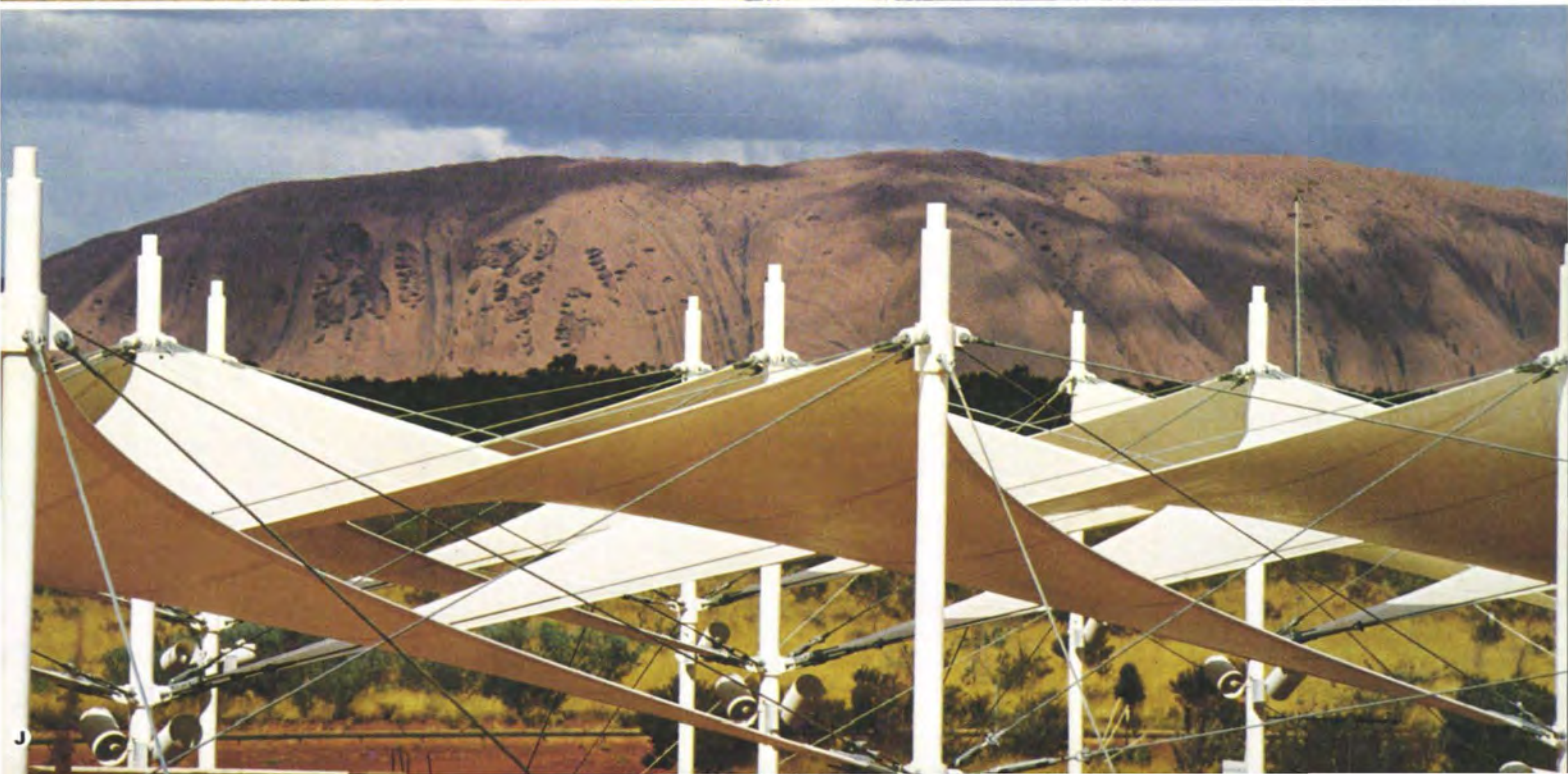
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Applications
Fabric roofs can now be found throughout the world covering:

Sports stadia, ice rinks, swimming pools;
Shopping malls and department stores;
Atria, indoor gardens and courtyards;
Leisure centres, theme parks;
Warehousing, industrial storage;
Cooling towers, radomes.

Coated fabrics are now being used extensively in hot countries to create shade.

The world's largest shade structure – the Haj Terminal at Jeddah International Airport covers 500,000m^2 and consists of 45m × 45m mechanically tensioned prestressed membrane surfaces again made from PTFE coated glass and reinforced with radial cables. The material's high surface reflection and the structural form helps reduce air temperature under the roof.

Large areas of sunshading have also been built in Libya, Saudi Arabia and Australia using PVC coated polyester material.

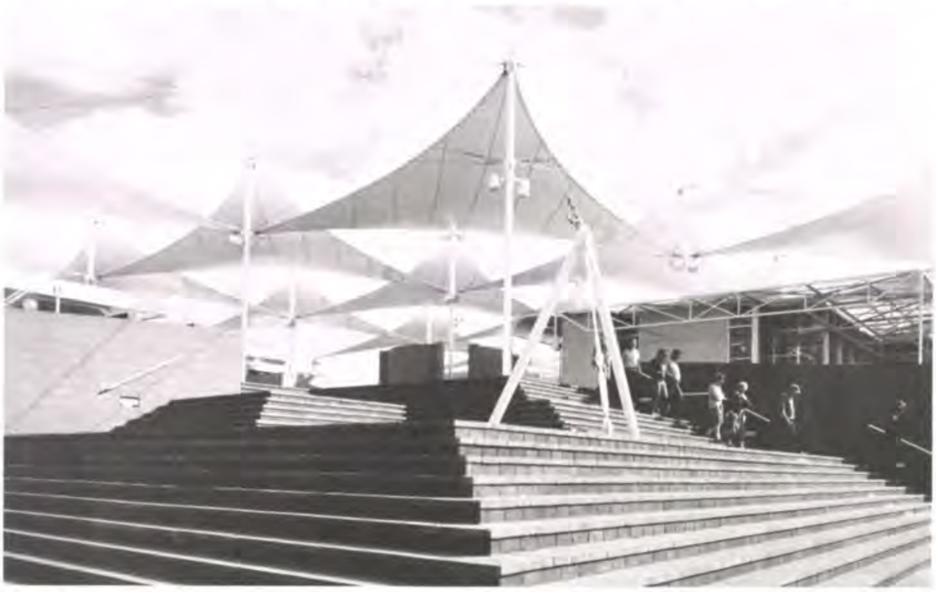


Fig. 6
Polyester membrane sails at Yulara, Australia

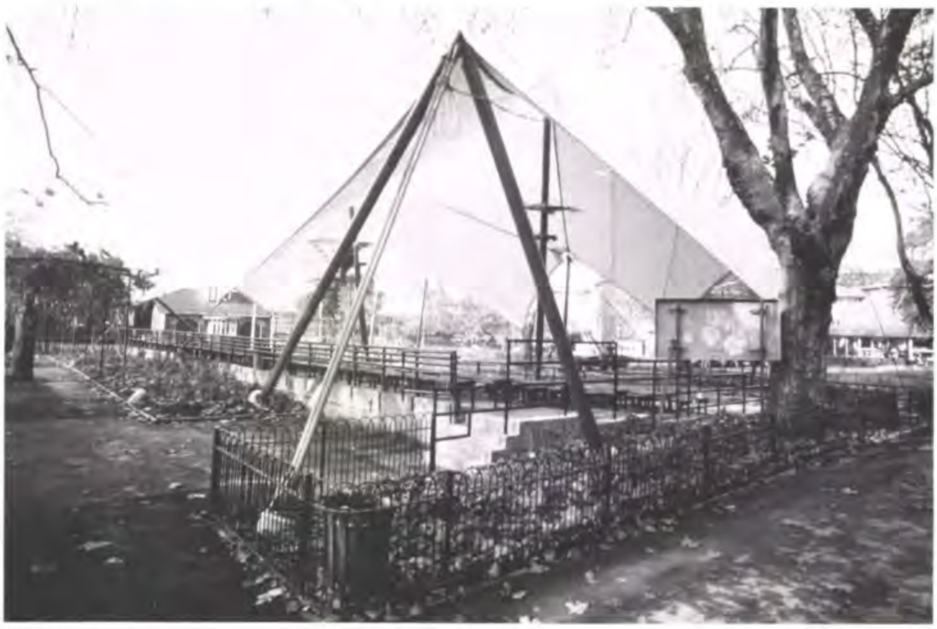


Fig. 7
London Zoo: PTFE coated glass canopy over Hummingbird Amphitheatre

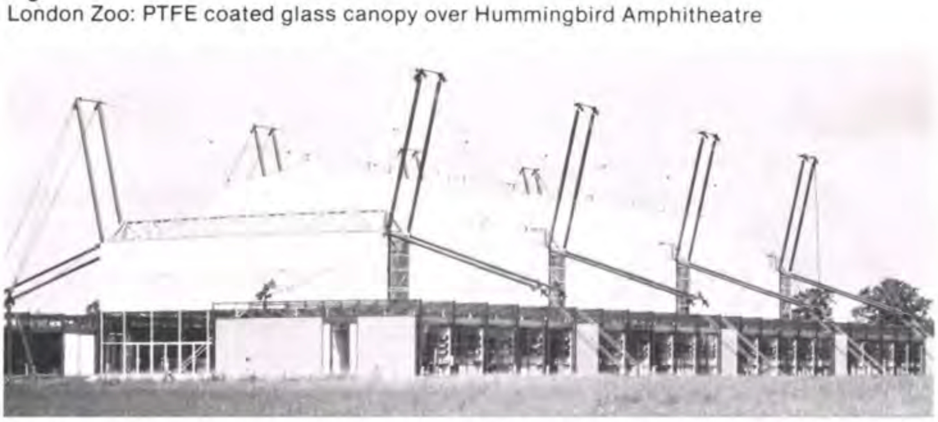


Fig. 8
Schlumberger Cambridge Research: PTFE coated glass membrane over laboratory

The decision to use a fabric structure can be for one of three reasons – image, cost, or some special property of the material, such as translucency or radio transparency.

The air-supported stadia roofs in Vancouver, Pontiac and Minneapolis have clear spans in the 200m range and surface areas of approximately 25,000m^2. These structures, made from PTFE coated glass, are reported to have been built more cheaply at this scale than conventional rigid roofs.

The Bullocks Department Store in San Mateo (6,500m^2) and several large shopping malls have also been covered with mechanically tensioned and prestressed membrane roofs made from PTFE coated glass. In some of these examples image is perhaps the dominant reason. Externally these buildings have a very positive architectural identity; internally the diffused natural lighting through the roof is very gentle upon the eye, contributes to a delightful ambience, and gives good colour rendition of goods.

At the Yulara International Tourist Resort[2] in Northern Territory, Australia, PVC coated polyester membrane sails, each 100m^2, have been used to give scale and shade to pedestrian routes and adjoining buildings (see Fig. 6). The total area covered is 12,000m^2.

In London Zoo a 600m^2 PTFE coated glass canopy covers a stage and terraced seating. (see Fig. 7).

Schlumberger Cambridge Research is a company dedicated to drilling and oil production leading to the development and application of new products. To reflect this attitude their new laboratories[3] are covered by a PTFE coated glass membrane covering 3,000m^2 (see Fig. 8).

At Mont Rouge in Paris, the same company have a pedestrian route between landscaped buildings. This route is covered with a 900m^2 PTFE coated glass membrane.

Materials
Clients, when considering a proposal which involves a fabric roof, ask questions about life, durability maintenance, fire, vandalism and cost. The answers to some of these questions come from looking at particular materials and how they have performed and, also, to a lesser degree, how they may be anticipated to perform.

To the building designer there seems at first sight a huge range of materials within the following categories – films, fabrics, and coated fabrics. For the great majority of applications it is not possible to use a film or a fabric on its own because films are not strong enough and because fabrics are not sufficiently durable and impermeable. It is therefore necessary to use a coated fabric or reinforced film. Coated fabrics are formed by taking a woven fabric of one of the following:

Glass
Polyamide (nylon)
Polyester
Aramid (Kevlar)

and coating it with one of the following materials:

Polyvinyl chloride
Silicone
PTFE
Polychloroprene *(Neoprene)*
Chlorosulphonated polyethelene *(Hypalon)*

Not all coatings can be used with all fabrics, and factors such as adhesion, chemical compatibility and coating temperatures need to be taken into consideration.

The general properties that are important in a building application are listed in Fig. 9 which also shows the relative dependence of these properties upon the coating and fabric. The two materials that so far have found success in building are:

PVC-coated polyester
PTFE-coated glass

A third material, silicon-coated glass, although only recently developed, promises to achieve just as much.

PVC-coated polyester

PVC-coated polyester has been used extensively in fabric structures of all types for the past 20 years.

Considerable improvements have been made during this time by some manufacturers in the formulation of their coatings, and a services life of 15-20 years can now reasonably be anticipated by using a relatively thick and densely pigmented coating.

White is considered to be the best colour because it reduces surface temperatures and this enhances life.

To maximize service life, translucency should be kept low. Light transmission, because of the thickness and density of pigmentation, should not exceed 6% through a single membrane. PVC coatings exhibit a dirt-pickup problem and unless regular cleaning is practised then light transmission is further reduced and appearance becomes progressively dull and unsightly. Thin surface coatings of Acrylic and PVF are applied by some manufacturers to alleviate this problem.

PVC-coated polyester is relatively easy to join by stitching or welding, and is easy to handle, transport and erect. Tight fabrication tolerances are not generally required because the material has a relatively low modulus.

The material can be produced flameproof to *BS 3120* but cannot satisfy any of the *BS 476* fire tests. However, several full-scale fire tests have been conducted on PVC-coated polyester structures and these serve to show that the material need not be a hazard.

PTFE-coated glass fabric

PTFE *(Teflon)* coated glass cloth has been used in many architectural applications in the USA covering sports arenas, department stores and shopping malls. There are also structures in Europe, the Middle East and Australia.

It is the first structural membrane material which exhibits qualities of permanence normally ascribed to more orthodox materials[4]. The PTFE coating is applied to the woven glass cloth by a dipping/sintering process which is repeated many times.

PTFE is an inert plastic material having a very high resistance to chemical attack and is unaffected by ultra-violet light. Dirt-pickup is not a problem and, although dust particles may rest on the surface, they do not permanently adhere and are generally washed away by the rain.

Structures erected 10 years ago in California still appear clean and white. There is no discoloration with age. Real-time weathering and artificial weathering tests indicate a service life in excess of 20 years.

Translucencies in the range 8% – 16% are available for outer skin structural membranes. Reflectivity of the surface is in the order of 70%.

The woven glass substrate makes the composite material difficult to ignite. It exhibits zero spread of flame in *BS 476: Part 7* and achieves classification S.AA in *BS 476:* Part 3.

Jointing of this fabric is done entirely by pressure heat sealing. PTFE-coated glass is inferior to PVC-coated polyester in terms of its crease resistance and so considerable care in handling, packing, and erection is required.

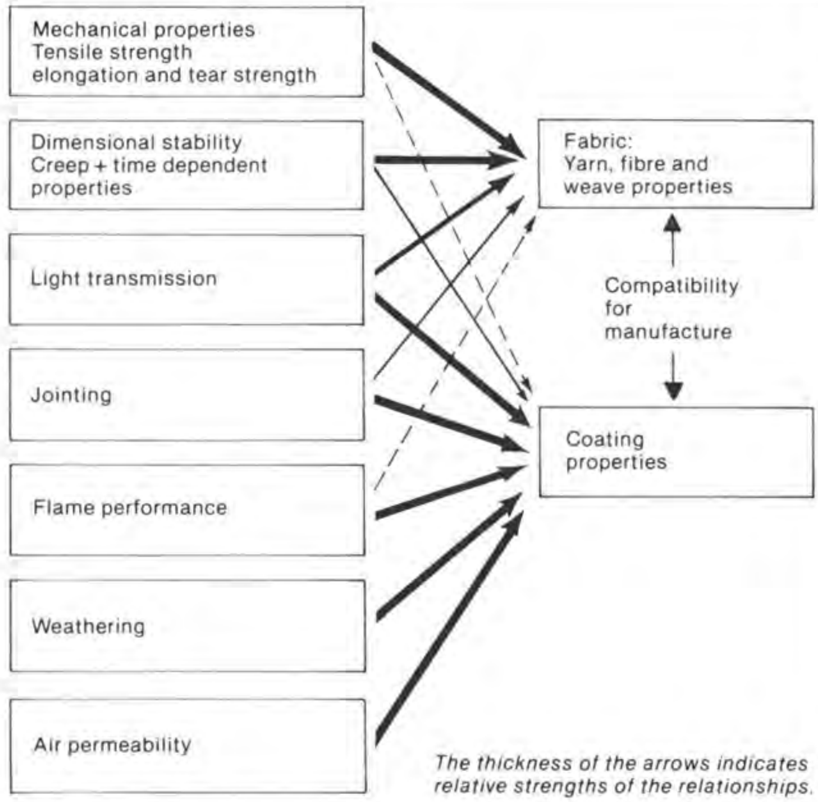


Fig. 9
Composite properties of coated fabrics and their relationship to the fabric and coating

Design goals

The process of designing a fabric roof involves the achievement of a number of goals:

(1) Defining the surface geometry
(2) Defining a workable prestressing method
(3) Analyzing the structure's response to loads
(4) Dimensioning of parts.

Surface geometry

Physical models have been the traditional means for developing surface geometry. They can be quick and unsophisticated and support geometry can be adjusted *ad hoc* to alter the degree and disposition of curvature throughout the structure.

This is the case with soap film models where surfaces of uniform tension can be formed within almost any set of boundaries. Each surface is unique to the particular geometry of its boundaries. Because the surface geometry is dependent upon the chosen support geometry of cables, masts, arches, etc., the refinement of the support geometry is a design task which should be completed before detailed calculations are undertaken.

Computer methods now exist which can numerically mimic the behaviour of soap films. Starting from the basic boundary co-ordinates, the computer calculates where prescribed prestress forces should lie in space for them to be in equilibrium. One such method called dynamic relaxation[5,6], is based upon the fact that to get from one state of equilibrium to another, a structure must move. By writing the equations of motion for the structure and applying damping to make the structure come to rest, one is following exactly the procedure that happens in nature. The method allows one to choose the loading conditions for defining the basic form which optimizes the structural behaviour. It is worth noting here that in contrast to orthodox building structures the weight of a coated fabric membrane is trivial.

Computerized methods avoid the serious measurement errors which can occur with physical methods. They also cope elegantly with cable reinforced membranes and other conditions where non-uniform tension regimes are desired.

However, physical models still retain the advantage of giving the designer physical understanding. Thus both methods are useful and complement one another.

Method of prestressing

There are two aspects to be studied. The first is to make a design which intrinsically can be prestressed. It is vital that a construction method and a method of introducing prestress is built into the design and detailing of the membrane/cable system from a very early stage. In the case of a mechanically tensioned membrane there has to be a system of parts which can be displaced so as to introduce, for instance, strains of 3%, for a glass fabric, along the weft yarns.

The orientation of the weave pattern must also be chosen with reference to the directions of principal curvature and the direction in which boundaries will be displaced. This implies for each structure a seam layout, and depending on size and curvatures of the structure, the density of the seams and thus the fabric width.

The second aspect concerns what forces are set up within the membrane during the introduction of the prestressing forces. A construction method, whilst having to be physically sensible, also has to be one which will actually achieve an acceptable distribution of prestress.

Construction methods can be proved in advance of the fabrication phase by using numerical methods. A computer model of the unstressed fabricated membrane would have applied to it the set of displacements that are intended in the real structure. The calculation yields the forces developed within the fabric elements as a result of

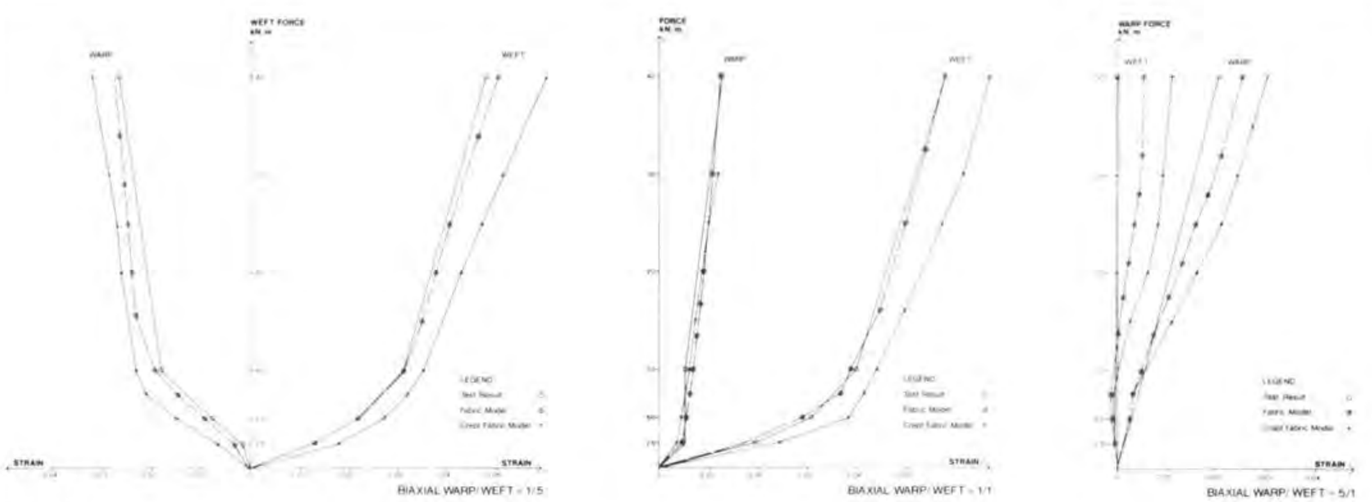
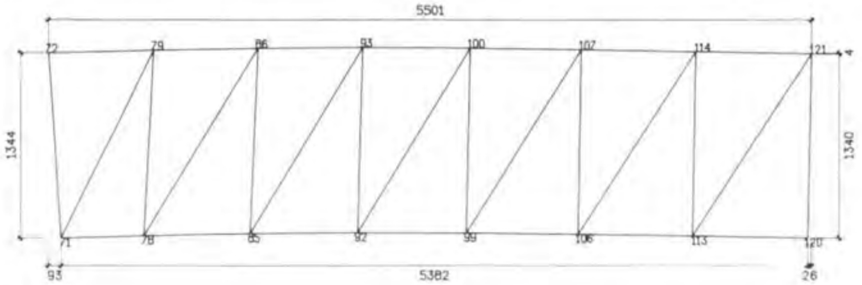


Fig. 10
Example of curve fitting based upon uniaxial and biaxial load tests



Cutting Pattern No. 13

Mesh Node No	Co-ordinates X	Co-ordinates Y	Layout Along	Layout Offset
72	-93	1344	0	0
79	661	1363	754	20
86	1415	1378	1508	35
93	2172	1385	2265	43
100	2946	1381	3039	40
107	3749	1369	3842	28
114	4573	1355	4666	14
121	5408	1340	5501	0
71	0	0	0	0
78	592	16	592	16
85	1362	31	1362	31
92	2136	38	2136	38
99	2921	36	2921	36
106	3727	27	3727	27
113	4552	14	4552	14
120	5382	0	5382	0

Fig. 11
Example of cutting pattern drawn and scheduled by computer

these displacements. On this basis the construction method can be developed. The computer's numerical model must cope with the non-linear, anisotropic, and creep characteristics of coated fabrics. Fig. 10 shows an example of curve fitting based upon uniaxial and biaxial load tests.

The achievement of prestress within a membrane can, in some circumstances, be critically affected by fabrication and construction errors. For instance, the supporting frame could be too large and the membrane too small. Also, the stiffness properties of coated fabrics can vary significantly within the material. The effect of dimensional and stiffness changes between different elements can be most effectively studied by numerical means, again in advance of construction.

In this way tolerances can be specified at several levels appropriate to different zones in the work.

Load response

Calculations have to be made which establish the strength characteristics required for the membrane material and its joints. Studies to find the critical load combinations during erection and service life have to be made. Snow and wind intensities can be established via codes of practice and wind tunnel testing.

A non-linear method of analysis, such as dynamic relaxation, must be used to allow for the large changes in geometry that occur under each pattern of applied loads.

Dimensions for fabrication

A feature common to all coated fabric membranes is that the surfaces are manufactured using patterns, as in dressmaking, such that when the parts are joined together they produce the doubly curved surface.

Traditionally this has been done by making accurate models of the surface, laying elements on this model and expanding the resulting patterns by a scale factor. However, having computerized the form finding and structural analysis phases of the design process, it became an inevitable step that cutting patterns could be drawn and scheduled automatically by computer (see Fig. 11).

Many considerations have to be taken into account at this stage as the final surface must be taut in the prestressed condition. This involves allowing for the prestress, anisotropic and non-linear fabric characteristics. Skilful juggling of the positioning of seams is required to avoid wastage of material through, for instance, banana-shaped panels.

Conclusions

Serious efforts are being made by manufacturers to produce high translucency membranes that are both strong and durable. Such materials could permit sports like soccer, hockey and polo to be played under cover, yet still on real turf.

If high translucency were to be combined with improved insulation so as to provide warm roofs complying with the building regulation requirement yet still transmitting 10% sunlight, then the total encapsulation of large shopping and recreational complexes within a single envelope becomes a real possibility.

Some current research[7] is directed towards the achievement of zero energy structures. These involve the use of multiple fabric skins having the effect of collecting and storing incident energy and re-using this to heat the internal environment when losses are too great. The development of responsive structures of this kind is one of the most exciting possibilities for the future, as it is completely consistent with the possibility of large spans. It seems certain that major advances will be made in the next 10 years and this kind of technology could have a profound effect on crop production in desert regions.

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Credits

A. All illustrations are copyright of Ove Arup and Partners.

B. Ove Arup and Partners designed the membrane structures shown in Figs. 6-8 with the following architects:
6. Phillip Cox and Partners, Sydney
7. John Toovey, FRIBA,
The Zoological Society, London.
8. Michael Hopkins Architects,
London.

C. In the case of Montrouge, Paris (Architects: Renzo Piano & Bernard Plattner, Paris), Rice Ritchie Francis were the consulting engineers to whom we gave computational support.

Armstrong Bridge

Michael Bussell

The Armstrong Bridge was opened on 30 April 1878, to replace the crossing of the Ouseburn via Benton Bank, then much steeper and narrower even than it is today. The greater part of its cost was met by Lord (then Sir William) Armstrong, and he and his works at Elswick were also responsible for its design, and the construction of the ironwork.

The event was reported in the *Newcastle Journal* next day:

'The inaugural ceremony of the opening of the viaduct crossing Benton valley took place yesterday morning. The bridge has been built under the joint superintendence of Sir William Armstrong, K.C.B., and the Town Improvement Committee of the Corporation of Newcastle...previous to the day of opening, the bridge was tested by the borough engineer, and found to be a substantial and complete structure. Although constructed of such substantial materials and at such a height above the ground, the bridge and its approaches present a very light and ornamental appearance...Sir W.G. Armstrong, addressing the Mayor, formally presented it, through him, to the borough, and said he was of the opinion that the readier means of access which it would afford to the very considerable amount of traffic passing between the borough and the neighbouring villages would be appreciated by all.'

Whereupon, having dedicated the bridge to public use, the Mayor with others present crossed the bridge before adjourning to a luncheon provided by Sir William at his Banqueting Hall in Jesmond Dene.

The idea

It would seem that the first idea for a high-level bridge to link Jesmond and Heaton came from Sir William Armstrong in 1875. At a meeting of Newcastle Council on 7 July that year a report from the Town Improvement Committee considered his proposal to erect an iron bridge at Benton Bank which would, by crossing the steep valley of the Ouseburn at high level, serve to form an improved communication with the Benton Road. The Committee felt that the project would form a most desirable improvement, would give great facilities to Jesmond, Heaton, and Byker, and open out the whole of the country to the north *(sic)* of the Dene, affording also an excellent access to and from Newcastle. It noted also Sir William's offer to pay the whole cost of the erection of the bridge, and of the formation of the new road leading from it to the Benton Bank but pointed out that the western approach through the property of the Virgin Mary Hospital would have to be formed at the expense of the Corporation. Parliamentary powers existed for such land acquisition, but the cost could be as much as £8,000 (less any return on acquired property which could be disposed of on completion of the work). The land for the eastern approach to the bridge from Benton Road, and the site of the bridge itself, were both owned by Sir William: they were included in his offer to the Corporation.

Sir William's outlay was estimated to be about £30,000, and the Council was therefore being invited to contribute less than one quarter of the capital cost of the bridge, but subsequently to accept the cost of maintenance once it was built. Although most speakers complimented Sir William on his specific generosity in this proposal, and deferred to his eminence, the overall impression from the record of the debate is

Fig. 1
Location plan showing the bridge and its approaches (from the 1:2500 Ordnance Survey Map, 1950 revision)

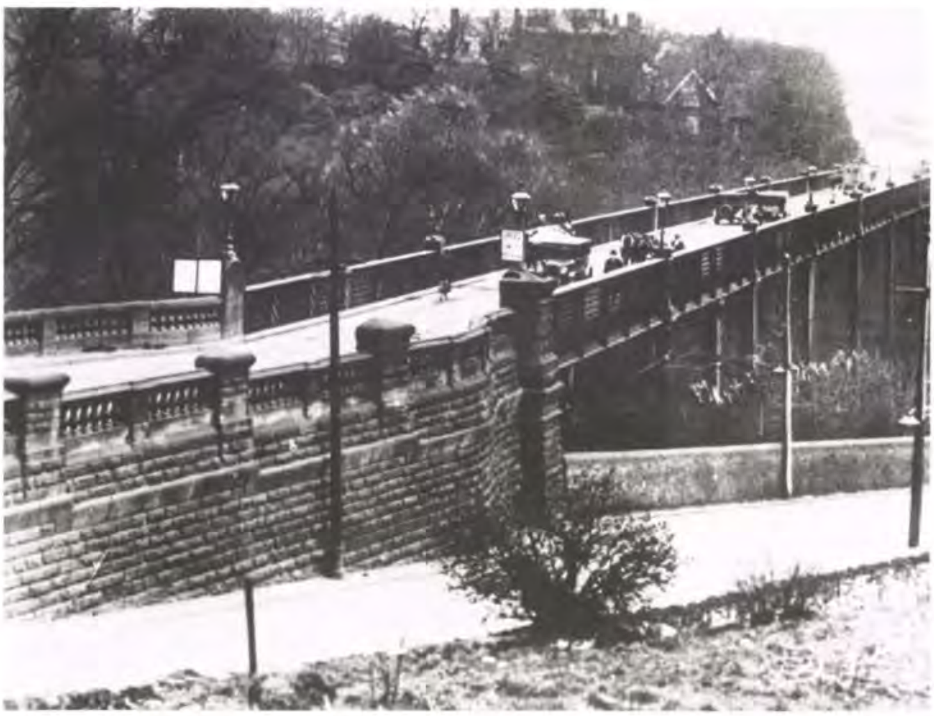


Fig. 2
Armstrong Bridge in 1926

Fig. 3
Aerial view of the bridge in the 1940s

that the Council would have preferred its windfalls to land on the ground with no strings attached!

After lengthy debate the matter was referred to the Finance Committee, which reported to the Council on 1 September 1875, favouring the proposal but recommending that the Council's liability for the scheme be limited to £5,000. A short discussion followed, in which it emerged that the bridge was to be free of toll. A motion to authorize the Town Improvement Committee to confer with Sir William Armstrong and make all requisite and necessary arrangements with him was approved, subject to the £5,000 limit on Corporation expenditure.

So Newcastle Council was to acquire a bridge across Jesmond Dene at only a fraction of the cost that would have been incurred had it been built and financed entirely by municipal enterprise. The community would benefit from an easier crossing of the Ouseburn than the narrow and steeply-graded Benton Bank could offer, and this improvement would open the way for Heaton to be transformed into a suburb of Newcastle, at a time when this area and neighbouring Byker were recognized as the only available locations for major new housing development close to the city centre.

As for Sir William Armstrong, his offer was but one of many generous actions towards Newcastle and its people, of whom at this time he was probably the wealthiest and the most famous. The Infirmary, The Hancock Museum, St. Nicholas' Cathedral, and Armstrong College (later the University of Newcastle) were only some of the public institutions to benefit from his generosity. And close by the bridge itself, both Armstrong Park and Jesmond Dene can today be enjoyed by the public as a result of the gift of these properties of his to the city.

It is true that he owned land in Heaton and could expect to benefit from its rise in value as building sites when the bridge improved communication between here and the city. But £30,000 was a much larger sum then than now and, taken with his many other benefactions, his offer must be seen as motivated principally by public-spiritedness. Nor was he seeking to build a monument to carry his name – that was unnecessary, for he was already world-famous as an engineer, gun-maker, and shipbuilder – and it was only after being opened that the Benton Bridge became commonly known first as Sir William's Bridge, and later as the Armstrong Bridge.

The bridge described

The bridge measured 552ft. between abutment faces, and the road surface varied in height from between 30ft. and 65ft. above local ground level. The eight wrought-iron spans were of equal length, and the usable width between the main lattice girders was 25ft., made up of a 5ft.6in. footpath on the south side, an 18ft. 9in. carriageway, and a 9ft. northside kerb.

The lattice girders were supported on abutments at either end of the bridge, and by seven pairs of square wrought-iron box-section columns resting on piers. These, like the abutments, were faced with buff sandstone.

Technically, a most interesting feature of the bridge was the way in which it was articulated to accommodate movements, and particularly those due to mining subsidence (which causes vertical and horizontal displacements in the ground) and temperature changes (leading to expansion or contraction of the ironwork). Each lattice girder was separately supported from its neighbour, resting on a cast-iron rocker bearing which pinned it to the abutment or column head. Similar bearings were attached at the base of each column, while two sets of x-braced wrought-iron tie-rods and a slender cast-iron strut linked the column pairs; these gave the bridge stability and resistance to wind forces, but were sufficiently flexible to allow some differential movement of the foundations. Turn-buckles on the tie-rods allowed for periodic adjustment to take up movements if these occurred. Finally, sliding bearings were provided at the central column, so that thermal expansion and contraction could take place freely here, rather than being countered by potentially damaging stresses in the girders or the abutments, as would happen if the entire structure was rigidly tied together.

The effects of mining subsidence and temperature change on the articulated structure are shown diagrammatically in Fig. 4.

The construction of a typical span is shown in Fig. 5.

The original road surface was macadamized (so named after its principal exponent, Sir John Loudon McAdam); this consisted of well-graded stones consolidated to form a wearing surface and cambered from 6in. thickness in the middle to 4in. at the kerbs. Beneath this, asphalt, again cambered but nowhere less than 2in. thick, rested on shallow-domed iron plates. These formed the structural decking, carrying loads to cross trimming girders 9in. deep and longitudinal girders 1ft. deep.

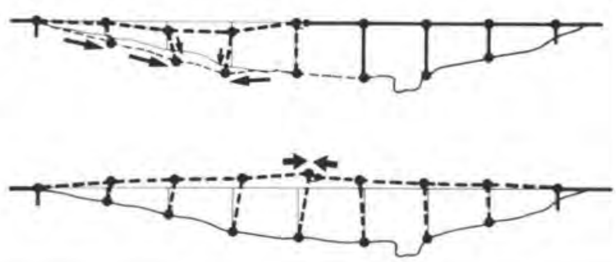


Fig. 4
Diagrammatic deviations of the bridge showing how the articulated structure can move to accommodate mining subsidence (top) and a rise in temperature (bottom)

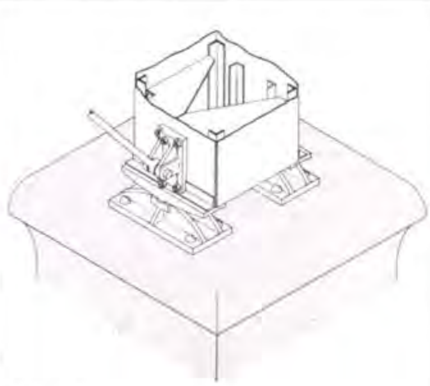


Fig. 6
Cut-away isometric view of the base of a column with the rocker bearing supports and the anchorage of a bracing tie-rod

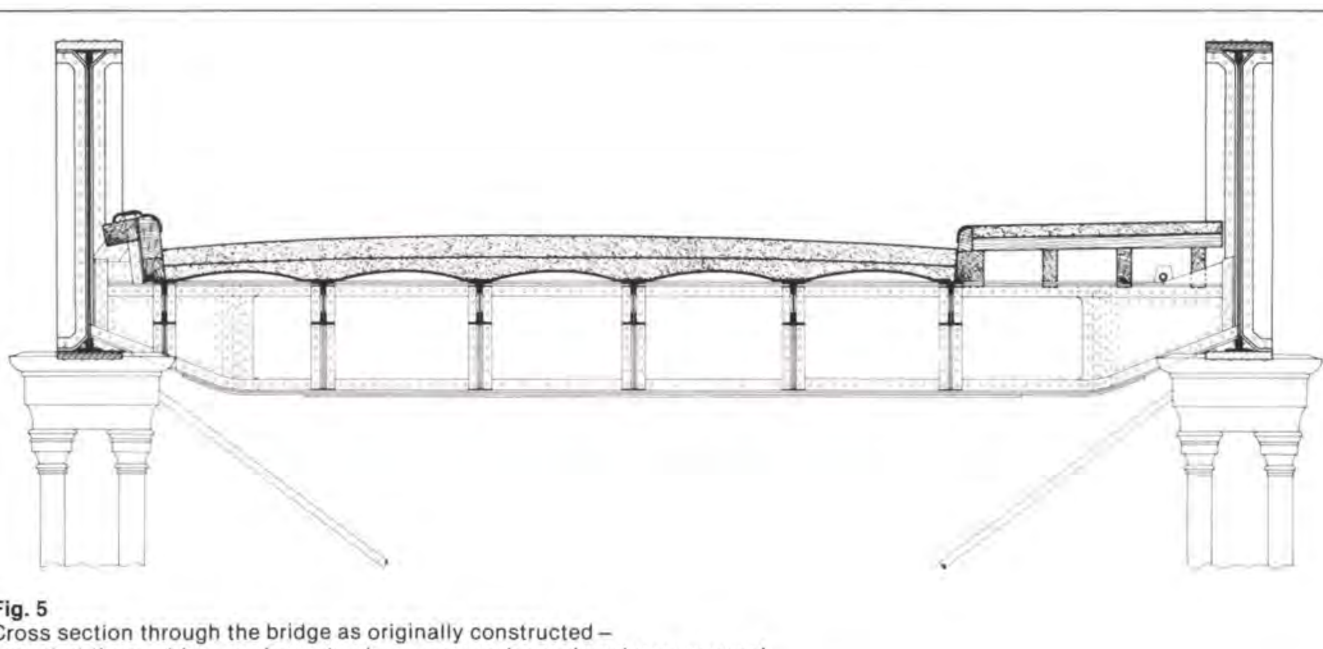


Fig. 5
Cross section through the bridge as originally constructed – note that the cast iron casings at column corners have since been removed

These in turn were supported by cross girders 2ft. 7½in. deep at 11ft. 6in. spacing. All of these girders were fabricated from wrought-iron plate and angle rivetted together. (It must be remembered that in the 1870s steel was still a novel structural material, its use in bridges being approved by the Board of Trade only in 1877, while the Armstrong Bridge was under construction. Equally, techniques for rolling solid iron joists had not advanced beyond the production of sections a few inches in depth, so that deeper beams had to be assembled using plates for the web and flanges, joining these together with angles and rivets. Welding as a means of connecting sections was still some years in the future.)

Use of latticed channels

The cross girders were carried by the main lattice girders, each 68ft. 11½in. long and 7ft. 3⅛in. deep at midspan. Here, the top and bottom flanges – which resist the bending action of the girder – were each made up of three wrought-iron plates 1ft. 6in. wide and ½in. thick. The latticed web was made up of wrought-iron channels 2¾in. wide. Their thickness varied, being greatest near the columns where the forces in the web are at a maximum. The use of latticed channels, as opposed to a solid web plate, is both practical in girders of this depth – where a thin plate would need frequent stiffeners to prevent buckling under load – and pleasing, as it gives the bridge a much lighter feel when seen from a distance and allows those crossing it to enjoy the view of Jesmond Dene.

The footpath was surfaced with asphalt on 3in. thick timber boarding, carried on 11in. by 4in. timber joists which span the cross girders. Like the roadway, the footpath fell towards the kerb so that rainwater could be drained away and discharged through the downpipe at the middle of the bridge.

The wrought-iron columns, like the secondary girders, were fabricated from plate and angles, rivetted together, having a square hollow section, the faces being of plate connected by continuous angles at each corner. Other angles and plates were provided horizontally at intervals to stiffen the whole assembly. The columns tapered, those in the middle of the bridge reducing from 3ft. at the base to 1ft. 9½in. width at a height of 35ft. 11in. Columns were linked laterally by 2in. diameter wrought-iron tie-rods, and the five tallest pairs also had a 5in. diameter circular hollow cast-iron strut horizontally near mid-height, supported by a vertical hanger from the diagonal bracing above.

Originally, the columns had a three-quarter-round-cast-iron casting at each corner. These were purely ornamental, perhaps an attempt to blend with the profile of nearby tree-trunks, but in practice they served only to trap rainwater which corroded the columns, and were therefore later removed.

At the head of each column, a cast-iron pilaster masked the junction of adjacent lattice girders. Those on the south side 'next to the footpath' also supported attractive cast-iron lamp standards, as did the abutment pillars at each end. Originally gas-lit, and fed by a 1½in. pipe under the footpath, these lamps carry their maker's name – 'W.T. Allen & Co., Lambeth Hill, London'. (The firm still trades as architectural metal workers, now in nearby Loman Street, but unfortunately has no record of this earlier contract.)

All of the ironwork and timber on the bridge, save only these lamps, was designed, drawn, and fabricated at Armstrong's Elswick works. The stone-faced piers and abutments (the latter considerably altered at the western end when the bridge was closed to vehicles in 1963) were undertaken by Messrs. W.E. and F. Jackson, contractors, of 4 St. Nicholas Buildings. This is almost certainly the firm that built the Ryhope Pumping Station Engine House in the late 1860s.

It would seem that Armstrong took some architectural advice when designing the bridge, as Mr. Rich's tracing dated 28 July 1876 is referred to on one of the original Elswick drawings in connection with levels of the piers and F.W. Rich was named (along with the Jacksons) as among those present when the bridge was opened. This local architect's later works included St. Gabriel's Church, Heaton and the Ouseburn School of 1893, neither far from the Armstrong Bridge, and it is quite likely that Armstrong would have sought counsel on aesthetic details from someone like Rich.

An essential consideration in any bridge design is the construction process and whilst no contemporary account exists of the building of the Armstrong Bridge, the sequence of operations can be surmized. Because of the articulation of spans and columns, it would be logical to start by construction of abutments and piers. Then the columns nearest the abutments could be set up and held in place by temporary bracing until the end-bay lattice girders were erected. These, like the columns, could conceivably have been assembled at Elswick and brought to the site on wagons, or alternatively made up in sections to be rivetted together on site. It is extremely unlikely that an entire span measuring 69ft. by 28ft. and weighing some 50 tons could have been put together at Elswick and transported to the site, so we must visualize the main lattice girders being erected, perhaps with the aid of Armstrong's own hydraulic jacks, and then receiving the cross girders and other secondary ironwork, the whole being rivetted together in situ by men working from a temporary timber decking.

The sequence could continue, with the next pair of columns being erected and braced to receive the next span of lattices, until the two ends of the bridge met at the central expansion joint.

The bridge in use

On 6 August 1879 the Council approved a motion that the seal be attached to an agreement between Armstrong and the Corporation, the bridge and its approaches all being completed: In effect, this meant that the bridge now belonged to the city.

Five years later, the bridge witnessed a most spectacular scene during the visit of the Prince of Wales (later King Edward VII), the Princess, and their children. Staying at Cragside as guests of Sir William Armstrong, they had arrived at Newcastle by train from Rothbury just before noon on 20 August 1884. Their day's programme included the opening of the new Natural History Museum, now the Hancock Museum (to whose construction costs Armstrong had contributed £10,000), preceded by the opening of Jesmond Dene to the public (Armstrong's land which, like Armstrong Park, he had donated to the city).

The procession of 41 carriages left Central Station and drove through packed streets to Armstrong Park, and thence crossed the Benton Bridge from east to west. The royal party received a most enthusiastic reception at the west end of the bridge. Although the crowd was very great at both ends of the Benton Bridge, and in the immediate neighbourhood, a large concourse of spectators managed to enjoy all the pleasure which the procession gave without risk of any personal inconvenience. At the Banqueting Hall in Jesmond Dene Road (included in Armstrong's gift of the Dene), an Address was presented by the Corporation to the Prince of Wales, who praised Armstrong's munificence. The Princess planted a sapling to commemorate their visit and, the Dene having been formally opened, the party proceeded to St. George's Hall, off St. Mary's Place, for lunch. The bridge settled back to a more prosaic routine, with its daily traffic of horse-drawn vehicles, handcarts, cyclists, and pedestrians. The next arrival that threatened to disturb this routine was the tramcar. This was already close at hand, for a section of the city's tramway system had been opened along Jesmond Road in December 1878. (It was to reach the Minories, still closer to the bridge, in August 1879). In January 1901 the chairman of the city's Tramways Committee proposed a widening of Jesmond Road up to the western end of Armstrong Bridge so that a double line of tramrails for extension over the bridge could be laid. No councillors at this time seemed to recognize that the bridge had not been designed to carry trams – or any other form of motor vehicle!

The City Engineer, however, was better informed. On 2 September 1903 the Council accepted his proposal that the tramway should instead run down Benton Bank, as the bridge was too narrow to accommodate a double line of tram tracks without disrupting other traffic, and a single line for two-way traffic would be equally inconvenient. The tramway would not come about for another decade.

Concern about corrosion

In the meantime, the City Engineer was becoming concerned about the condition of the Armstrong Bridge. He reported in 1906 that on four of the 14 columns (each carefully inspected):

'Very extensive corrosion has occurred where the wrought-iron work has been covered by these (cast-iron) ornamental castings, owing to the water having found its way and lodged there. One column has several places which have been entirely eaten away and through which daylight can be seen. In the other three columns examined the corrosion has been very serious but the plates are not quite eaten through. The City Engineer considers it a fair assumption that the remaining ten columns are likely to be in a similar condition...the riders and flooring have not corroded much because of constant inspection and several repaintings.'

The City Engineer estimated the cost of repairs to the columns at £2,700 and strongly urged the removal of the ornamental castings. He also doubted whether this cost was justified on a bridge already too narrow for existing traffic, and added (perhaps gratuitously) that the bridge was of very inferior design and that no amount of repairs could remove some of the very objectionable features, although he chose not to say what he thought these to be.

In 1907, the castings were removed from the columns. At the same time, the permissible loading was reduced from 12 to 6 tons with a speed limit of 5 m.p.h. £50 was spent on immediate repairs – perhaps the net difference between outlay and recovered scrap value on the decorative, but damaging, castings.

To facilitate the tramway which was to bypass the bridge, the City Council, in October 1912, approved the widening of the stone-arch bridge at the foot of Benton Bank and its approaches, from 22ft. 9in. to 50ft. overall. In addition, the gradient was to be eased to a maximum of 1 in 12 from its then 1 in 9. The tramway across Benton Bank was opened on 6 June 1913, and led to the further development of Heaton as a suburb – something that the opening of the Armstrong Bridge had already triggered, although not perhaps as extensively as may have been expected. Unhappiness continued to be expressed about the inadequacy of the Ouseburn crossings at Benton, and in 1926 the Council considered the City Engineer's report on alternatives for a wholly new route across the Dene. He noted that Benton Bank was unsatisfactory as a main road and particularly so for trams, as these were subject to speed restrictions and compulsory stops, and the gradient of 1 in 12 on either side of the burn made for even slower running. As for the Armstrong Bridge, it seemed in generally good condition, although three columns were again badly corroded and some main girders showed signs of overloading. Weight restriction notices had been fixed at each end of the bridge, but as no watchman was employed there was no supervision to prevent traffic over 6 tons crossing the bridge. (At this time, the Scotswood Suspension Bridge – whose narrowness and condition were also giving concern – did have watchmen).

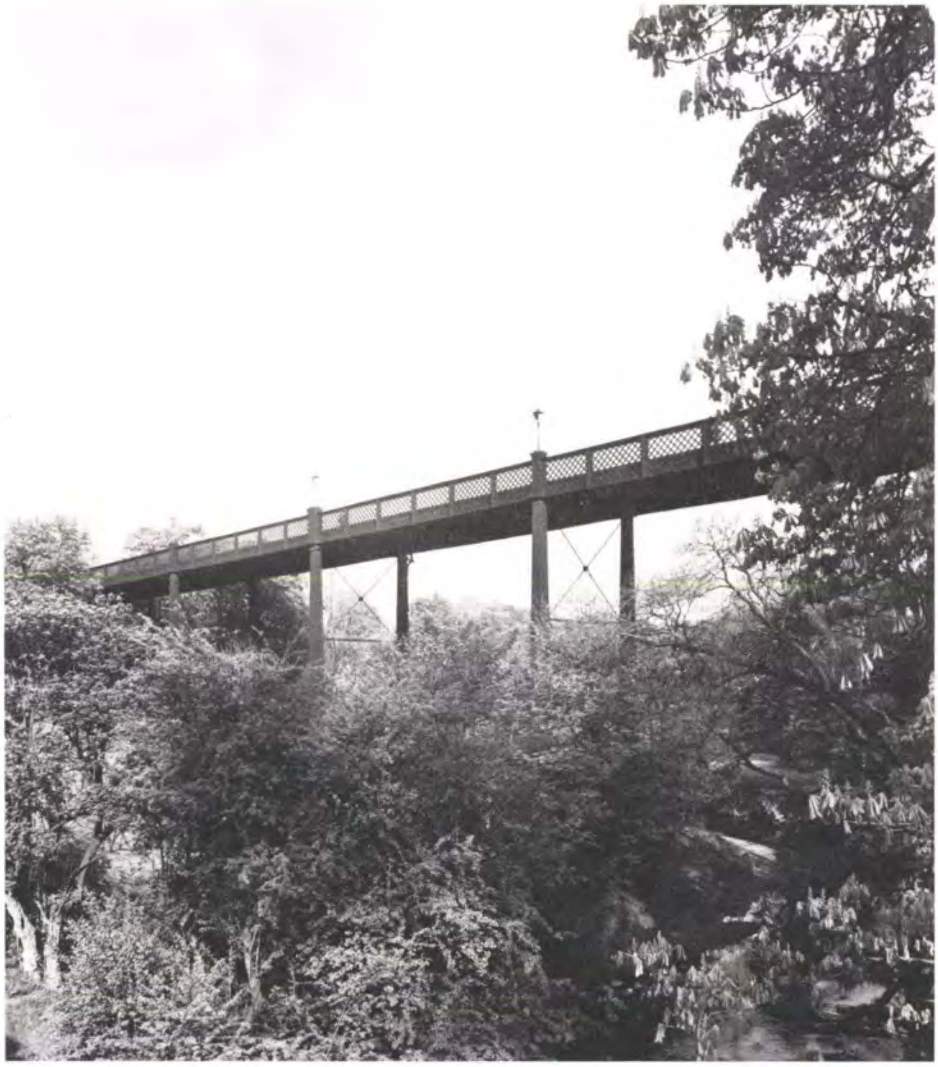


Fig. 7
The Armstrong Bridge:
view from the forest floor

Fig. 8
View of the Armstrong Bridge
from the abutment level

(Photo: Henk Snoek)

The City Engineer proposed to by-pass the present crossing with a new road just north of the bridge, 60ft. wide and graded at just 1 in 168. Four options were considered: an embankment that would effectively block the Dene and views along it (estimated cost £190,000), a steel bridge (£325,000), a masonry viaduct (£406,000 - praised for its aesthetic qualities but deplored for its high cost!) and a reinforced concrete bridge (£292,000, but condemned by one lay councillor as involving the use of solidified mud).

In the event, a decision on this proposal was deferred, one councillor (Alderman Sir George Lunn) observing let the people 40 years hence get a bridge for themselves! His compromise motion was passed, to make the Armstrong Bridge safe for passenger and light vehicular traffic, other vehicles being obliged to use Benton Bank.

He, for one, recognized that the bridge had been designed at a time when road traffic was lighter, both in weight and volume: 'let them try and use the Bridge for the purpose for which Lord Armstrong designed it.'

The bridge seems to have received little Council attention until 1956, when the weight limit was reduced from 6 to 2 tons, but despite new warning notices, many heavier vehicles continued to take the shorter, easier alternative to Benton Bank. The merging of traffic from the two routes also caused jams and delays at each end, so the Town Improvement and Streets Committee resolved finally that the bridge should be closed to all traffic and that Benton Bank should be raised to ease the gradient. This was not to happen until 1963.

In the meantime, in 1960, urgent repairs had to be made when it was found that timbers supporting the footpath had rotted.

The bridge was finally closed to traffic on 17 July 1963 following the regrading of Benton Bank and other work to improve the footways. Although the original closure order limited its use to pedestrians only, in practice cyclists have continued to take advantage of this convenient and undemanding alternative to Benton Bank; they, like those on foot, are still benefiting from Armstrong's gift.

Recent developments

As has been seen, the rapid growth of road traffic prompted thoughts of a new crossing of Jesmond Dene as early as 1926 and the idea often re-emerged over the years. In the 1960s, when Newcastle Council led by T. Dan Smith was laying plans for a city of the future, a Tyne Coast Expressway was proposed. This would have involved a new dual-carriageway, Jesmond Dene Bridge, whose alignment would have necessitated the demolition of the Armstrong Bridge. At that time, public interest in the bridge seemed minimal, but this began to change when it acquired a new function, that of an open-air arts and crafts market. Organized by the Novocastrian Arts Fair, the first exhibition took place on Sunday 27 July 1969. Initially, four Sunday shows had been agreed by Newcastle Highways Committee, but public support was so good that these have continued and grown into an institution almost as traditional and as valued as its precursor along the Bayswater Road in London. In both locations, trees and parkland form a backcloth that enhances the setting. Meanwhile, the Expressway scheme has lapsed.

Early in 1980, Tyne and Wear County Council, who had become responsible for the bridge in 1974, published, for public consultation, a statement on the condition of the bridge, and proposals for alternative solutions to the problems this posed. The bridge had recently been surveyed, and extensive corrosion had been found in the secondary girders under the roadway and in the columns. Wet rot was present in many timbers, some of the cast-iron pilasters over the main girder supports had cracked, and the central expansion joint had locked solid as a result of corrosion, so that movements caused by temperature changes were no longer being taken up by free expansion at the joint. Instead, these movements were pushing and pulling on the abutments and damaging them so that the western abutment needed shoring.

The statement recognized the popularity of the arts fair which often attracts large crowds, and warned that these could continue for only about five years if no major repairs were done. If the fair was stopped and the usable width of the bridge narrowed to 10ft., it might be 10 years before repairs became essential, although in any event some rotted timbers under the footway would have had to be made good.

Cost of major repairs

Major repairs to the bridge (by then listed Grade II in the Department of the Environment list of buildings of architectural and historical merit) could cost £170,000. With additional expenditure of £25,000 on repainting the bridge every five years, it was felt the bridge's life could be extended another 25 years.

Alternatively, a new concrete footbridge might cost £250,000, although this would not be wide enough to accommodate the arts fair. To construct a replica of the existing bridge would be much more expensive, maybe £600,000. A fourth option noted that pedestrians and cyclists, who use the bridge in fair numbers throughout the day, might perhaps be catered for by a new footpath away from Benton Bank (and implied presumably that the Armstrong Bridge could subsequently be dismantled, although this, and the costs arising, were not stated).

Tyne and Wear County Council called for comments from the public on these proposals, which were exhibited at the Newcastle Central Library in February 1980, and held a public meeting at the People's Theatre, Jesmond. From these, from letters in the local press, and from a petition with over 5,000 signatures, it became clear that public opinion was overwhelmingly in favour of retaining the bridge.

The arguments were not based only on the bridge's age and historical associations: it was seen as an attractive structure and the opportunity was not lost to point out how many of Newcastle's historical buildings and structures had been pulled down in recent years. Not least significant was the view that, apart from its attraction as a setting for the arts fair, the bridge still served a useful purpose – a crossing of the Ouseburn between Jesmond and Heaton safer and more convenient than the Benton Bank alternative, especially for cyclists and those on foot, heavily laden or pushing prams.

Plan of action

As a result the County Council formulated a plan of engineering action which included repairs to the west abutment and replacement of the wrought-iron columns.

When finance was made available permanent repairs were undertaken. New slide bearings were installed at the west abutment and in order to ensure that all temperature movement would occur at the position of the new bearings and also to maintain stability during construction, the existing central expansion joint was permanently clamped whilst the east abutment was tied back to reinforced concrete anchor walls. Once partial reconstruction of the west abutment had been completed, a new reinforced concrete bearing shelf and plinths were constructed. The new steel sliding bearings were positioned on flat jacks and were raised into position and grouted in place before removal of the temporary props. The interiors of the original wrought iron columns were totally inaccessible for maintenance and, consequently, internal corrosion must have commenced not long after construction in the often humid internal environment. The Department of the Environment granted permission for the gradual replacement of these, by tapered I section columns manufactured from steel plate.

The work on the bridge, financed by a grant under the Derelict Land Act and carried out by the Cleveland Bridge and Engineering Company Ltd., was started on 10 January 1983 and completed 12 weeks later at a cost of just over £132,400. During column replacement, temporary repairs had to be effected to the pier wind bracing. New wind bracing members will be constructed in the near future.

Whilst work on the bridge was being carried out, the County Council commissioned the manufacture of replacement panels and pilasters. The joints between adjacent longitudinal beams, coincident with column positions, were originally masked by ornate cast-iron panels and top flange pilasters. These had not weathered well and at the time of column replacement very few were still in position. A new set was commissioned and placed in store for future use.

Further repairs will be phased over several years to allow for replacement and/or repair of wrought-iron deck beams, replacement of the existing timber footway with reinforced concrete, grit blasting and painting of the whole structure, and erection of the beam end panels and pilasters. A sum of £30,000 per annum is to be spent on the bridge until repairs are complete: the work is expected to take four to five years. Modifications will be necessary but these will either be hidden or will appear similar to the original. The ultimate aim is to return the Armstrong Bridge to as near to its original form as possible, and to retain not only a tangible reminder of Newcastle's past but also part of our engineering heritage.

References and acknowledgements

The two principal documentary sources are the original bridge drawings (of which copies were kindly made available to the writer by the Newcastle City Engineer in 1968), and Minutes of meetings of the City Council and various of its Committees, held by the County Archivist. Two leaflets on the future of the bridge were published by Tyne and Wear County Council in 1980.

As yet, no other primary sources of information have come to light despite the efforts of many who in 1968-70 searched their records at the writer's request, in particular: the late Lord Armstrong at Cragside; the staff at the Newcastle Central Reference Library and the Northumberland County Record Office; Mr. J. Hewitt and others at Vickers Ltd.; Frank Atkinson of the Beamish Open-Air Museum; the Radio Times – Houlton Picture Library; W.T. Allen & Co., London, and Stafford Linsley.

This paper appeared in the November 1984 issue of In Trust, *published by the Tyne & Wear Industrial Monuments Trust.*

Details of Trust membership and other publications are available from the Trust at Sandyford House, Archbold Terrace, Newcastle Upon Tyne, Tyne & Wear (0632 816144, Ext. 291).

Jesmond Dene Bridge

Bill Smyth

Ave

This sad story is a footnote to Michael Bussell's article. It is sad because it might have been the other way round, and because an enormous amount of work and care went into producing an exceptional design which will never be built.

Newcastle-upon-Tyne in the 1960s was in the throes of motorway building. The dual three-lane expressway from Newcastle to the coast was to cross Jesmond Dene on a bridge, leaving the existing road down Benton Bank in operation but requiring the Armstrong Bridge and the Robson Theatre to be demolished. We were appointed by the City in 1966 to design the bridge and our preliminary design report was produced in February 1967.

Atque

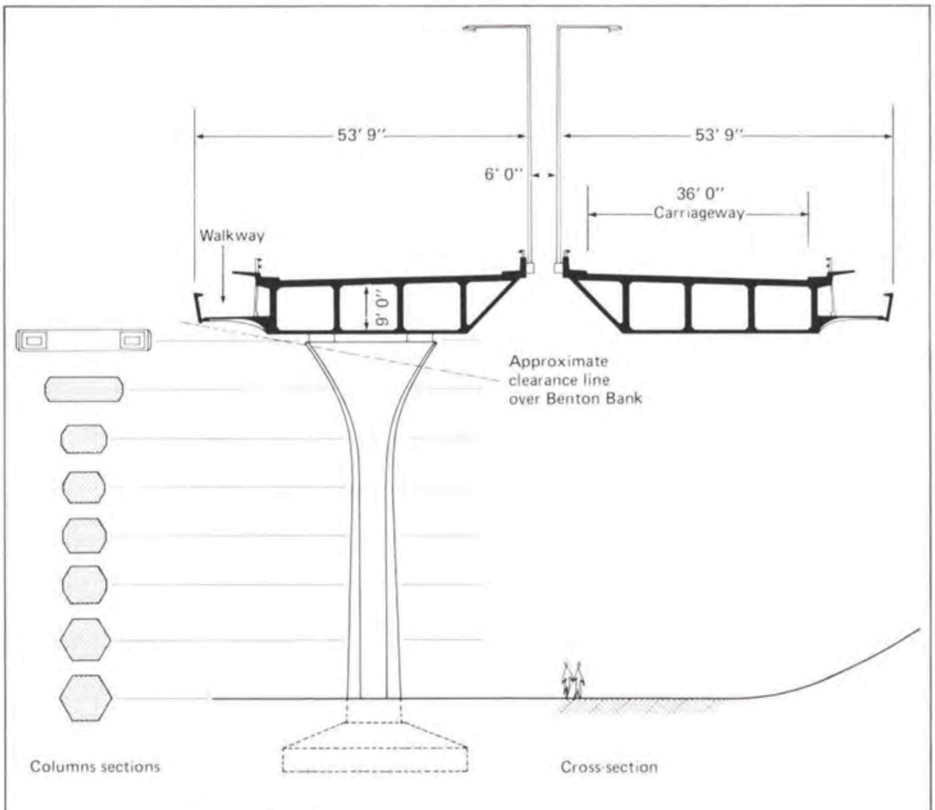
The design is an interesting one. The two carriageways are on separate decks to let light down between them and to reduce the effect of the enormous width (we have been doing this wherever we could, ever since). The piers carrying the two decks are staggered to avoid the various obstacles on the site (road, stream, footpaths). The arrangement is orderly though; the inner spans are all equal and the three pairs of piers are staggered by the same amount.

The two decks are multicellular prestressed concrete boxes. They are very deep; the span to depth ratio is about 17:1. This is economical and it is not apparent, because what is seen at the edge of the deck is a walkway with a much smaller visible depth.

The walkways are cantilevered from the edge of the deck. They are well below the road level, so that pedestrians are completely protected from direct noise and splash and can look out over the valley in relative peace. They connect to underpasses incorporated in the abutments at about the same level. From these underpasses, steps lead down in the middle of the abutments, connecting to the footpaths on Benton Bank and in the valley.

The piers are of varying heights. They had to be designed to be made in the same formwork and to look alright whether long or short. The shape is basically related to the forces the piers would have to carry but a lot of development with models was required to arrive at the final version.

The bridge was to be made of dark concrete to tone with the trees in the same way as the black-painted Armstrong Bridge does. This may have been wishful thinking.



Jesmond Dene Bridge: Section

Vale

The design was accepted by the client and by the Royal Fine Art Commission. Detailed design started, so did objections to the expressway; enough roads had been carved through Newcastle to produce a backlash.

An appeal was made to the Ombudsman which caused a delay of a year. The Ministry of Transport had the verges increased in width, which required some redesign, and the walkways were redesigned to improve the appearance and convenience. Finally in 1972 the bridge went out to tender. A price within the estimate was received from a good contractor. The contract was about to be awarded when someone found an error in the Order Plans. The job stopped and never restarted.

The road has remained as one of Newcastle's worst bottlenecks. There have been 116 accidents in the past 10 years; four people have died and 24 more have been seriously injured. The Tyne & Wear County Council has started design work on a road on a similar line to the former proposal, but much reduced in standard; presumably the Armstrong Bridge will be left intact, but the park in the valley will be spoiled. I refrain from drawing any moral.

Jesmond Dene Bridge: Models
(Photo below: Henk Snoek)

The Arup Ideas Competition

Martin Manning

For some years the Department of Industry and the Off-shore Futures Club have been carrying out a study on man-made islands. The central figure in all this has been Professor John Allen who was until recently Professor of Aeronautics in Cranfield and known to many of us within the firm.

During a discussion at the Off-shore Futures Club of the possible uses of man-made islands it was suggested that one might be as a permanent, if peripatetic, home for the Olympic Games. The possible advantages of this would be to not only reduce the investment required each time to stage the games but also possibly to depoliticize them.

Professor Allen had learnt some time ago of our involvement with the Pilkington Glass Age projects and in particular with Sea City. He therefore drew the obvious conclusion that possibly Ove Arup & Partners would be the right people to talk to about such an idea.

From that conclusion was born this year's Ideas Competition.

We were lucky to obtain the help of Emlyn Jones who prepared a brief for us for the sporting accommodation. Thus in the latter part of last year the competitors set off.

Some 20 or 30 submissions were received. The range offered was wide. They can best be classified by the grain, and thus mobility, of the primary structure. The coarsest grain of primary elements involved single floating structures on which all of the activities of the stadium should be located.

The first refinement of this was to construct the single floating stadium from a number (approximately 15) of smaller identical units. It was thought by most that these would be easier to transport. Various proposals were made for ways in which the units could be levelled and connected.

The next refinement was to reduce the number of primary elements to two or three and to provide secondary structure to infill between them. In this case several entrants proposed the use of redundant supertankers as the primary elements. The mobility of such a solution is self-evident.

Two types of infill structure were considered. The first type involved conventional bending structures supported by the primary units. The second proposed some kind of continuous surface floating on the surface of the sea. Such solutions included ice layers as well as sand, with negative pore water pressure, placed on an inflated mattress.

The final class of solution proposed that the stadium element should be located on a submerged sand island which would have to be constructed in each location each time.

Most entrants considered that the stadium would be protected from the action of major waves. However some did propose that wave actions could be used to generate power required in the stadium.

The Judges

A description of the competition and its results was included in an Arup appendix to the report prepared by the Off-Shores Futures Club which was submitted to the Department of Industry. The Chairman of the judges for the competition was Professor Allen. He was joined by Nick Grimshaw, one of our architect friends, and our very own Michael Shears. We are very grateful to them. After a lengthy session of judging the judges decided that prizes should be awarded as follows:

- First Prize, Robert Stewart
  of Building Engineering Group 6
- Second Prize, John Harvey
  of the Birmingham Office
- Third Prize, Frank Pyle
  of Building Engineering Group 5

Additionally the judges felt that three further entries deserved a commendation. They were from:

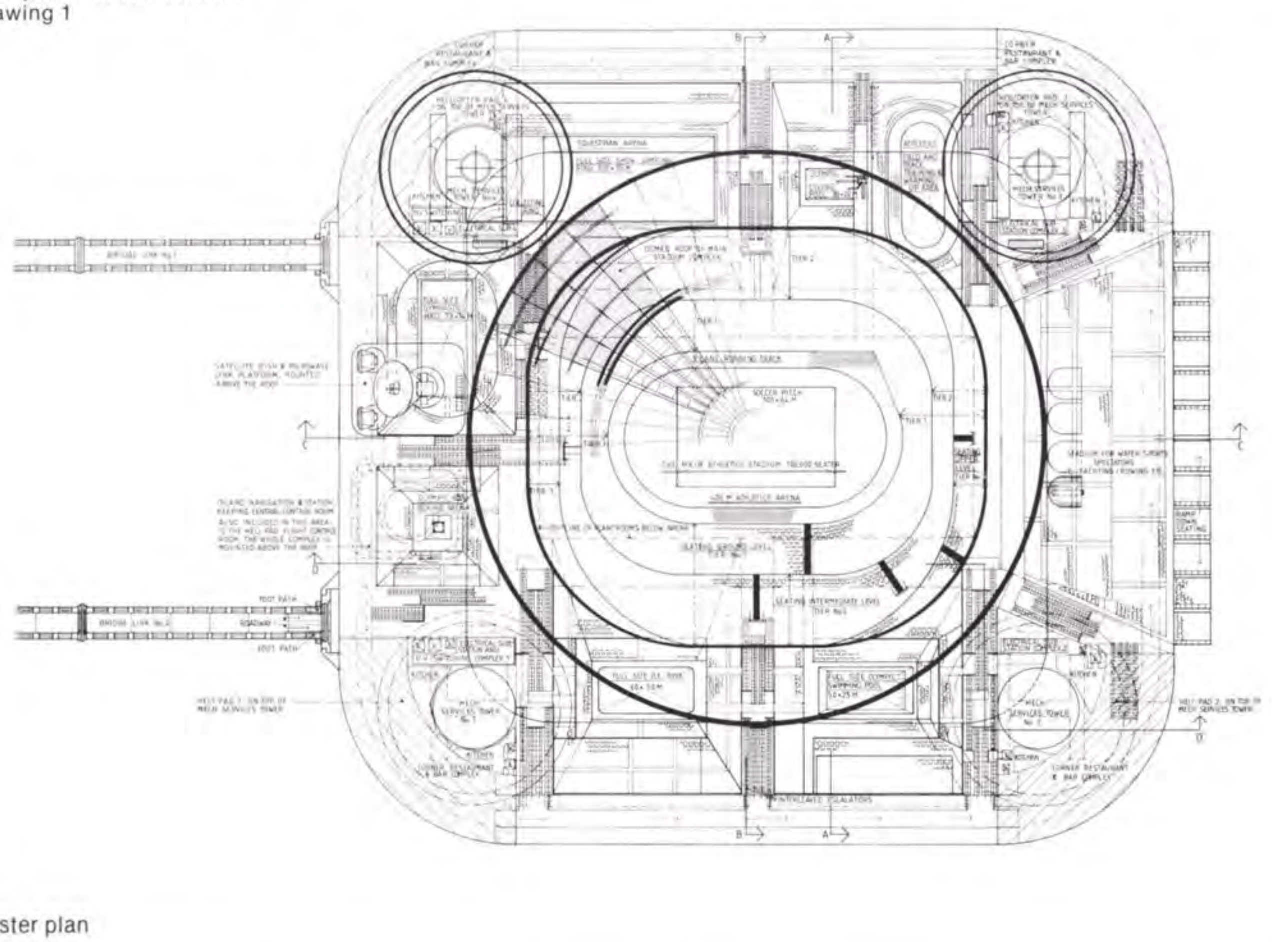
- Ian Smith and Richard Waters
  from Building Engineering Group 5
- David Badger
  from the Birmingham Office
- Alistair Hughes
  from Building Engineering Group 6

I think you have to draw your own conclusions about the consistency of the origin of the successful entrants. The judges were particularly impressed with Robert Stewart's entry which had obviously involved an enormous amount of work. It included not only drawings, some of which are reproduced here, but also a report touching upon nearly every aspect of such a problem. Congratulations and admiration are due not only to Robert but to all the prize winners and those commended.

Of the man-made islands report itself more is expected within the future. It is likely that a Man-Made Islands Association is to be formed under the umbrella of the Major Projects Association. This will be housed in the Oxford Management Centre.

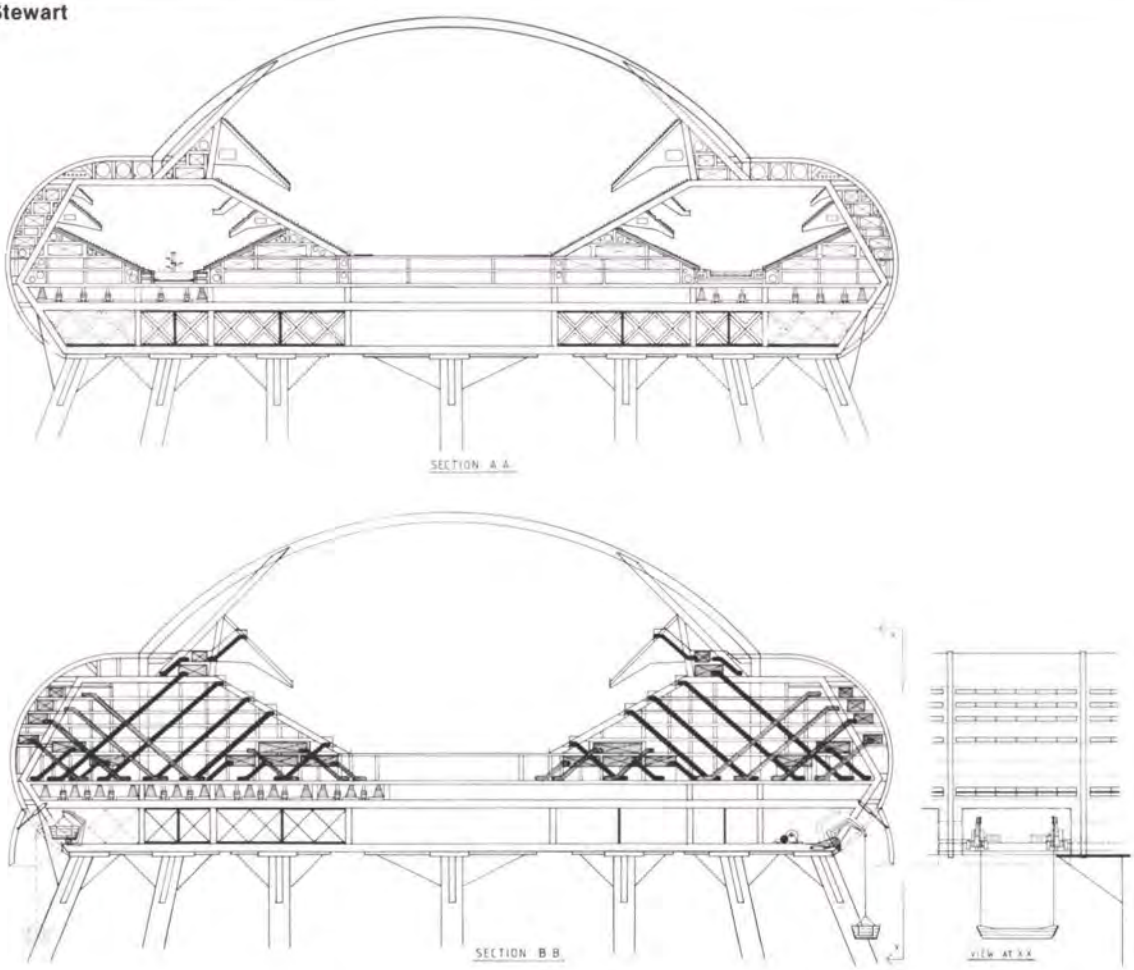
Of course we must let the Birmingham Office build their Olympic Stadium first but after that we will look forward to receiving our appointment to design and supervise the construction of the permanent floating home for the Olympics.

First prize: Robert Stewart
Drawing 1



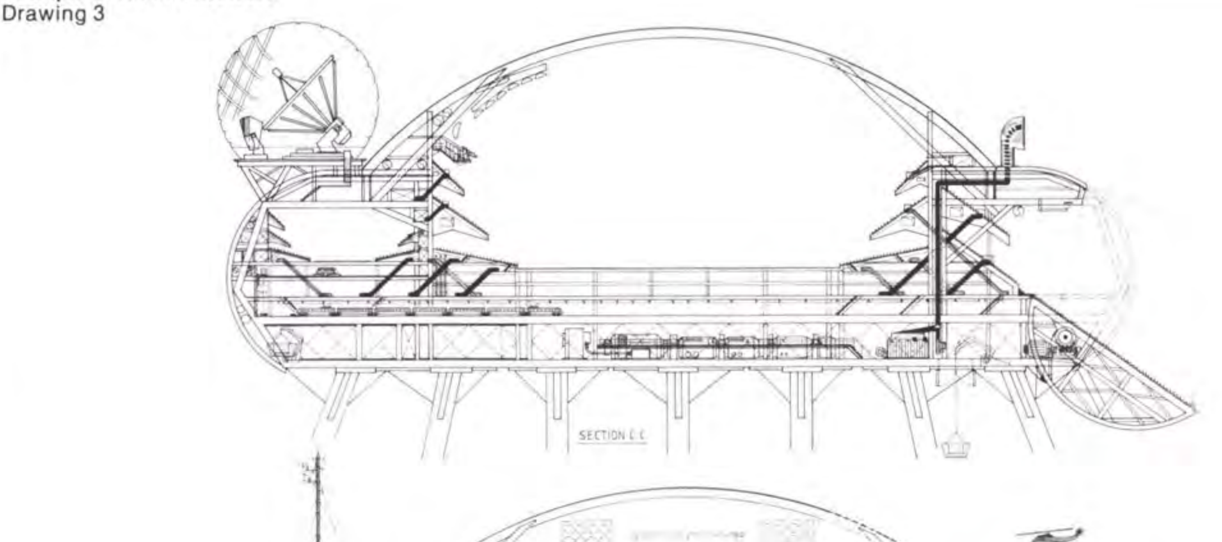
Master plan

First prize: Robert Stewart
Drawing 2



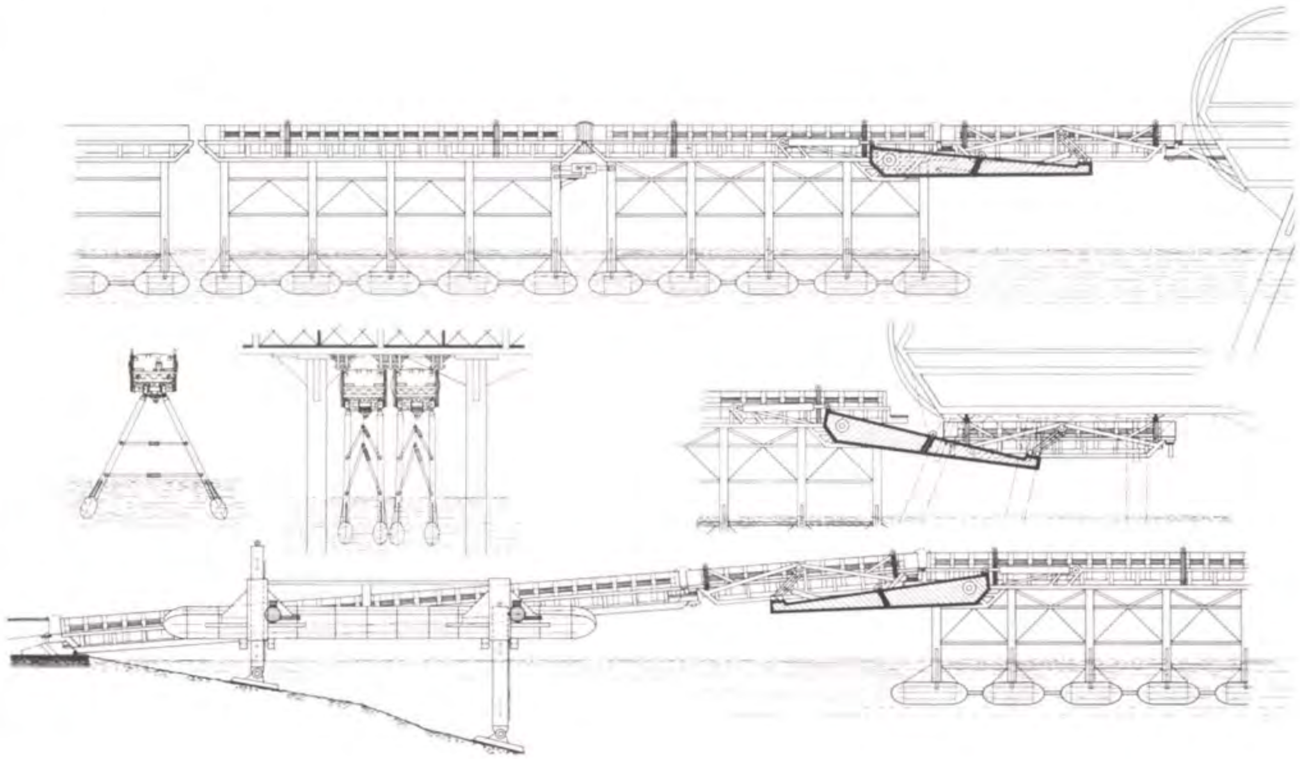
Section A – A and Section B – B

First prize: Robert Stewart
Drawing 3



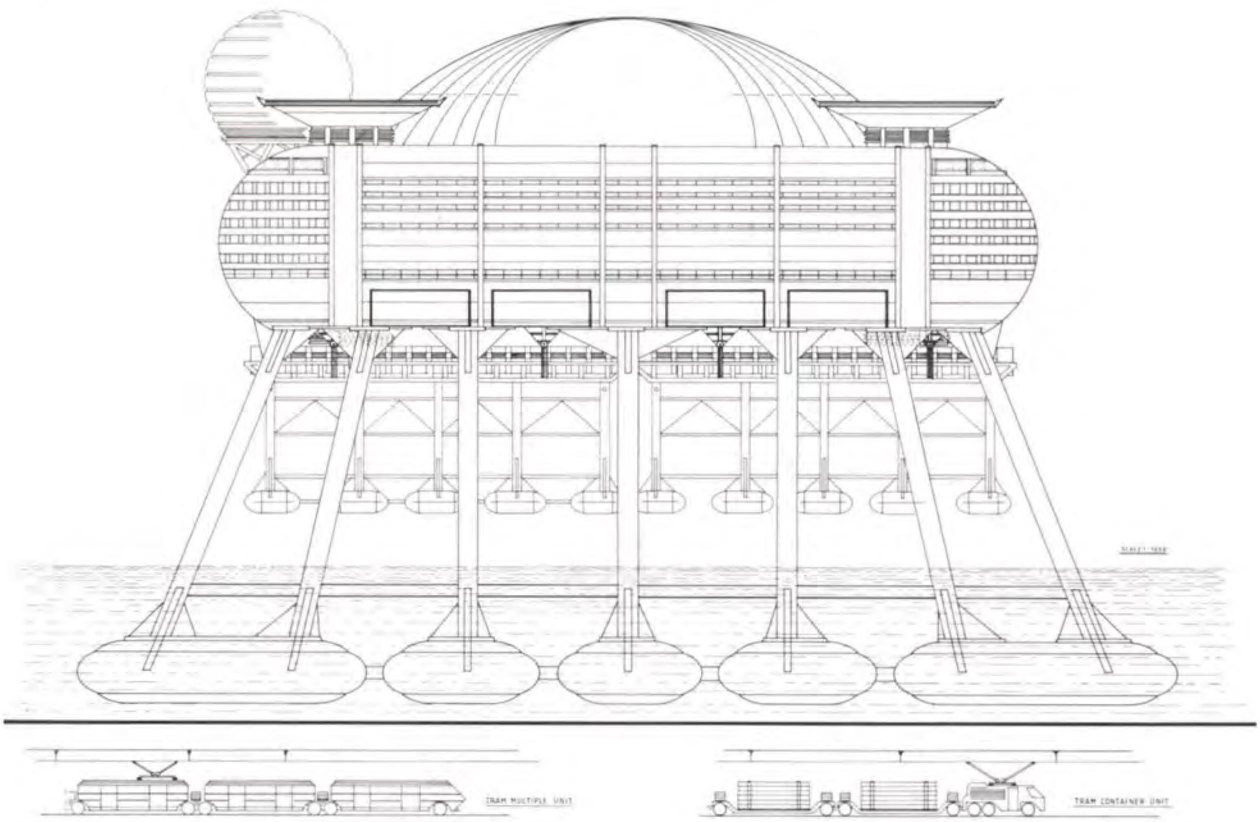
Section C – C and Section D – D

First prize: Robert Stewart
Drawing 4

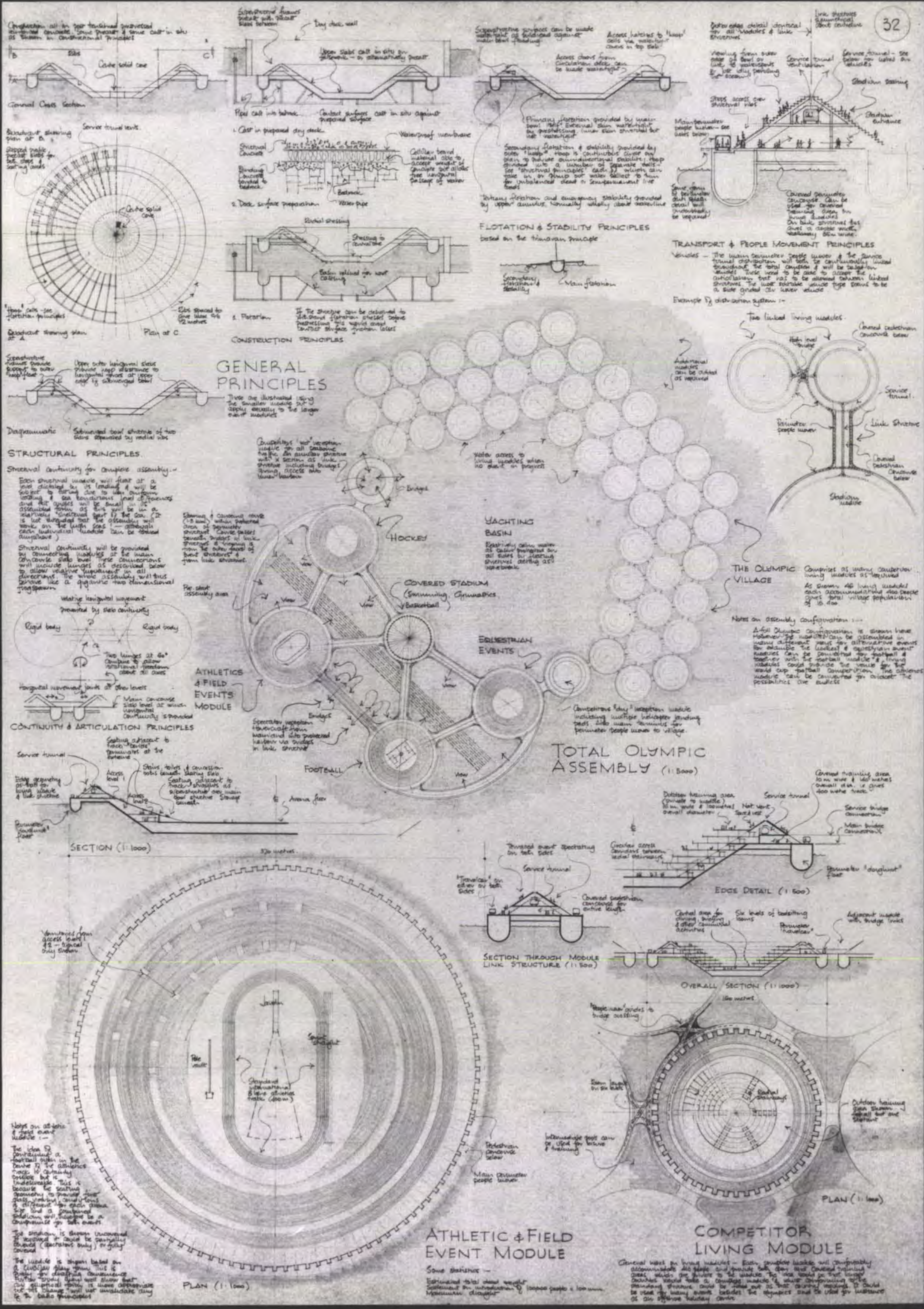


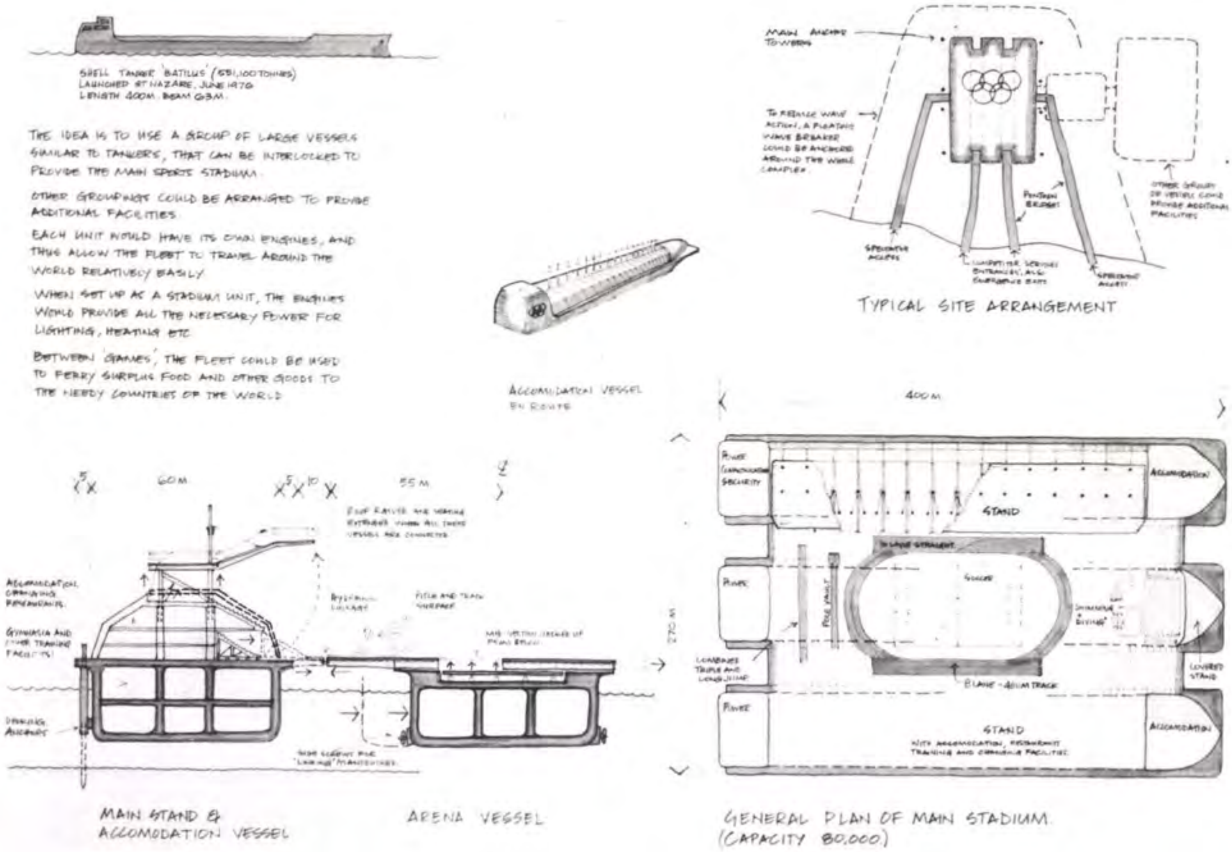
Bridge link details

First prize: Robert Stewart
Drawing 5



Island elevation and miscellaneous details


GENERAL PRINCIPLES
These are illustrated using the smaller module but apply equally to the larger event modules
Conical Cross Section
Centre solid core
Plan at C
CONSTRUCTION PRINCIPLES
FLOTATION & STABILITY PRINCIPLES
based on the trimaran principle
Main flotation
TRANSPORT & PEOPLE MOVEMENT PRINCIPLES
Two linked living modules.
Service tunnel.
Link Structure
Stadium module
Diagrammatic
STRUCTURAL PRINCIPLES.
Structural continuity for complete assembly.
Rigid body
Rigid body
CONTINUITY & ARTICULATION PRINCIPLES
HOCKEY
YACHTING BASIN
COVERED STADIUM
(Swimming, Gymnastics, Basketball)
EQUESTRIAN EVENTS
ATHLETICS & FIELD EVENTS MODULE
FOOTBALL
Bridge
View
THE OLYMPIC VILLAGE
Notes on assembly configuration :-
TOTAL OLYMPIC ASSEMBLY (1:5000)
SECTION (1:1000)
Service tunnel
Arena floor
EDGE DETAIL (1:500)
SECTION THROUGH MODULE LINK STRUCTURE (1:500)
Service tunnel
OVERALL SECTION (1:1000)
PLAN (1:1000)
ATHLETIC & FIELD EVENT MODULE
Some statistics :-
PLAN (1:1000)
COMPETITOR LIVING MODULE



Second prize: John Harvey (left)

Third prize: Frank Pyle (above)

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