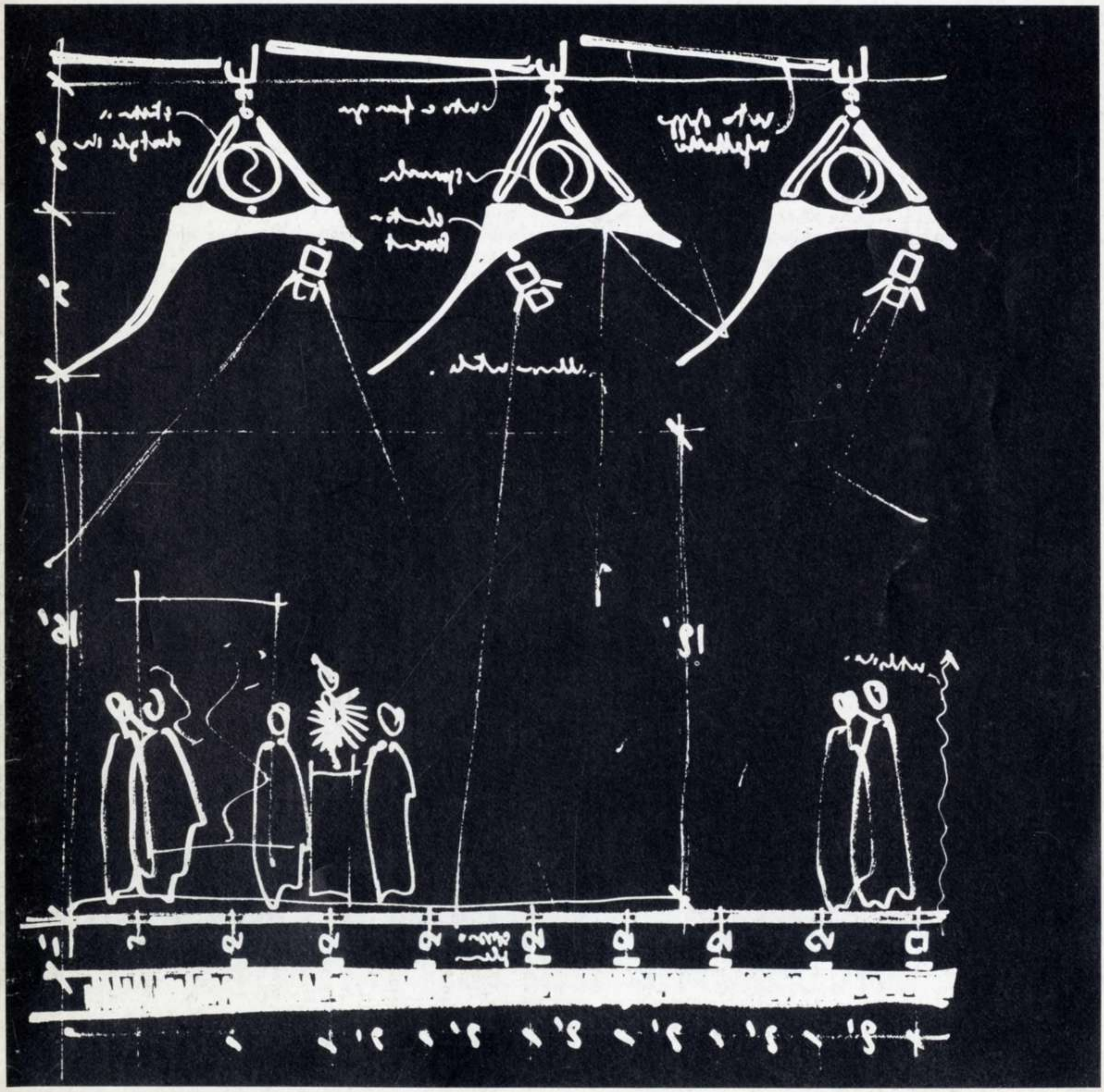
THE ARUP JOURNAL

APRIL 1983



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The Menil Collection, Houston, Texas

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In 1981, the Menil Foundation commissioned Renzo Piano to design a permanent home for the Menil Collection of Art and Historic Artifacts. It is to be located in one of the older residential suburbs of Houston, surrounded by traditional timber frame houses, and near the campus of St. Thomas' University.

During visits to museums and galleries in Europe and the Middle East and in subsequent discussions, Madame de Menil and Renzo Piano evolved three specific ideas regarding the type of building most suitable to house the collection:

(1) The buildings should be in harmony with their environment and non-monumental in scale. The setting should be of a domestic nature, including garden areas, and one or two of the existing timber 'Balloon Frame' houses on the site.

(2) Since the complete collection, currently housed in Houston, New York and Paris, is too large to be put on permanent display, a significant part of the collection should be housed in a storage area. This area should provide secure safes, with environmentally stable conditions, and should also allow occasional viewings by art scholars. These safes, called the 'Treasure House', should be prominently located for all to see since they house the major part of the collection. Works kept in the Treasure House will, however, be displayed for limited periods in the public galleries.

(3) Exhibits in the galleries should be viewed under natural light coming predominantly from above. In addition, this natural lighting of the galleries should reflect any changes in the weather and the time of day.

The fundamental architectural concept for the collection is the platform roof. This forms a unifying element, covering the whole building, pulling together all aspects of the design. Located below the platform will be the galleries, gardens, and external and internal walkways. The platform, acting like an umbrella, will provide protection from the external environment, and will also allow the effects of any changes in this environment to be felt. In particular, the platform roof will reduce the level of natural light into the spaces below, without any means of mechanical control. It will be formed of an assembly of specifically designed, precision made, structural elements, capable of performing this role, as well as supporting any additional components. Located prominently above the platform roof, and linked to the ground floor through the roof, will be the Treasure House.

Roof design development

Development of the design for the building, including structural and services requirements, took place during design sessions at Renzo Piano's studio in Italy. Due to its overriding importance in the project, the greatest proportion of the design time was spent in the development and detailing of the platform roof and its elements. At the outset, it was necessary to establish what functions the roof should perform, and four main areas were identified, namely:

(1) Light control
(2) Solar gain control
(3) Structural performance
(4) Additional components.

(see Fig. 5)

Based on international standards, a level of 150 lux was adopted for lighting a permanent display of art work. This standard gave an allowable annual exposure of 3×10^6 lux hours and formed the basis of the design.

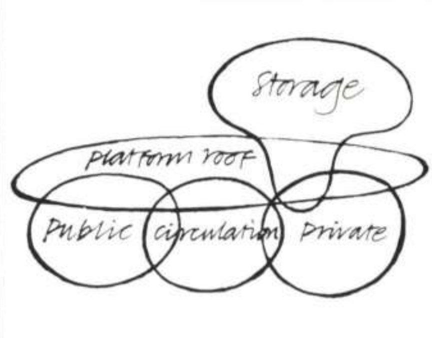
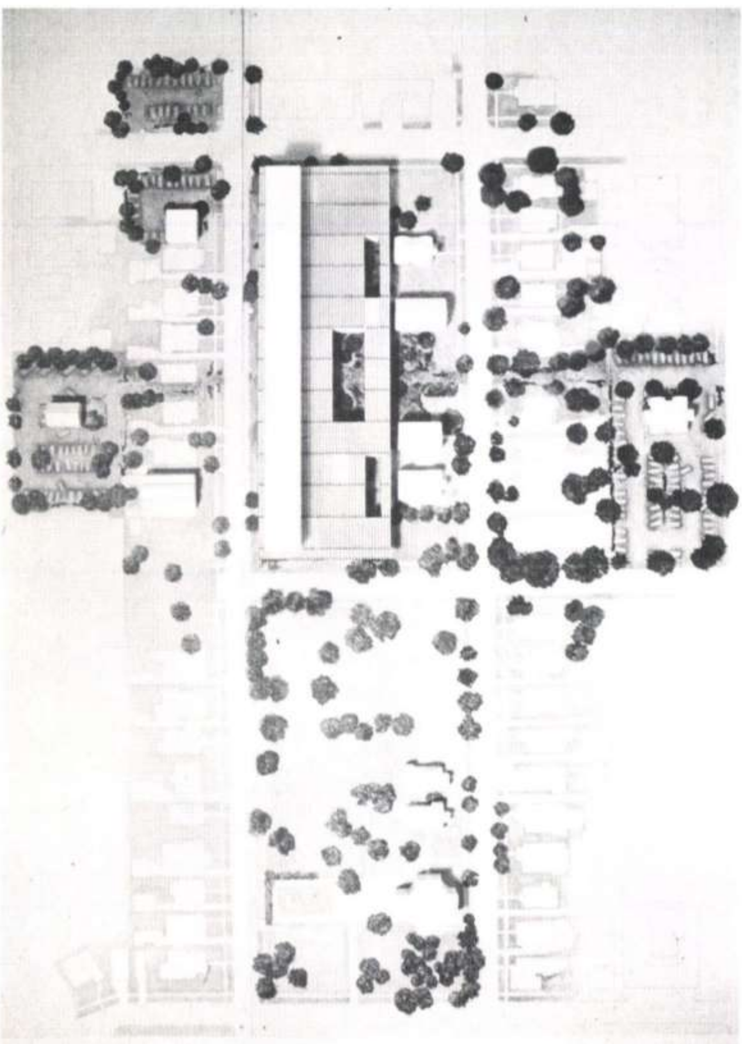


Fig. 1
Piano's basic idea

Fig. 2
The unifying element

Fig. 3
The site (Photo: Bel-Air Photo)

Fig. 4
Model of the Menil Collection Building



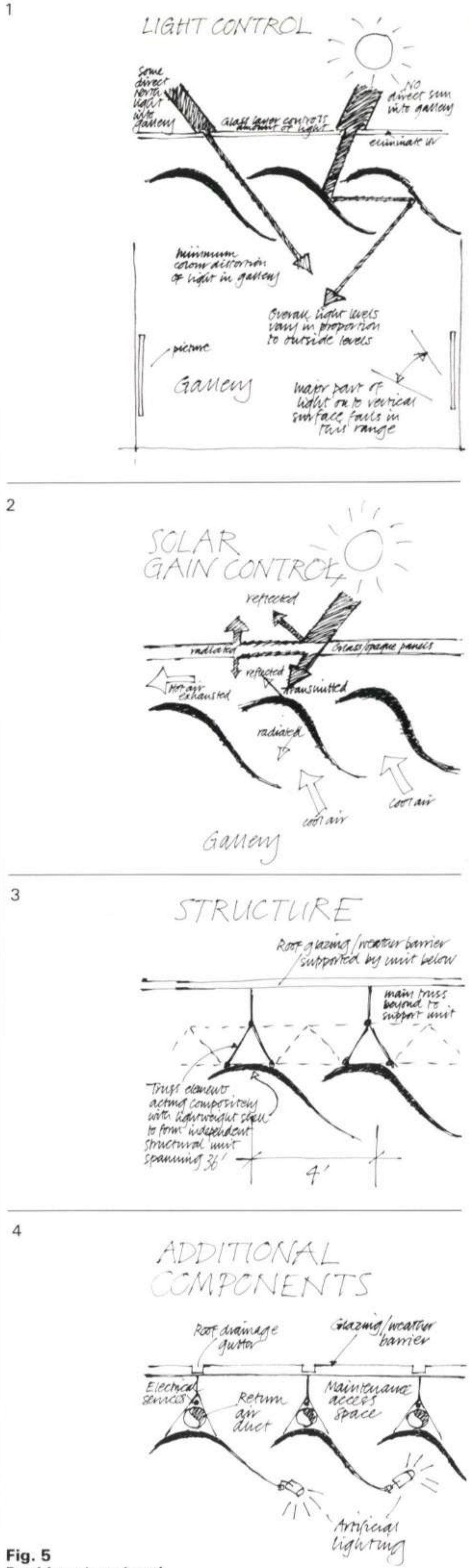


Fig. 5
Roof functions 1 to 4

Since the works, being kept for the most part in the Treasure House in darkness, are only exhibited for short periods in the galleries, the level of light on the paintings during these periods can be higher than 150 lux provided all ultra violet light and direct sunlight are excluded. To achieve this level of light, a system of reflecting elements and roof glazing was developed. A glass of the right light transmittance value, acceptable on colour performance, and containing an ultra violet filter, was required. The reflecting elements should prevent all direct sunlight from entering the spaces below, but should reflect this sunlight, and the brightness of the whole sky, into these spaces.

The shading function of the leaves also assists the solar gain control by reflecting the heat to outside and forming a barrier above which the heated air collects. This led to the principle of supplying cool air at floor level, and extracting air at roof level. This is achieved by forming a pressurized air supply plenum below the gallery floor, and allowing the air to distribute through linear, low velocity grilles at near room temperature. This also allows room temperature and humidity to be accurately controlled. The primary means of reducing solar heat gains, however, is the type of roof glazing used.

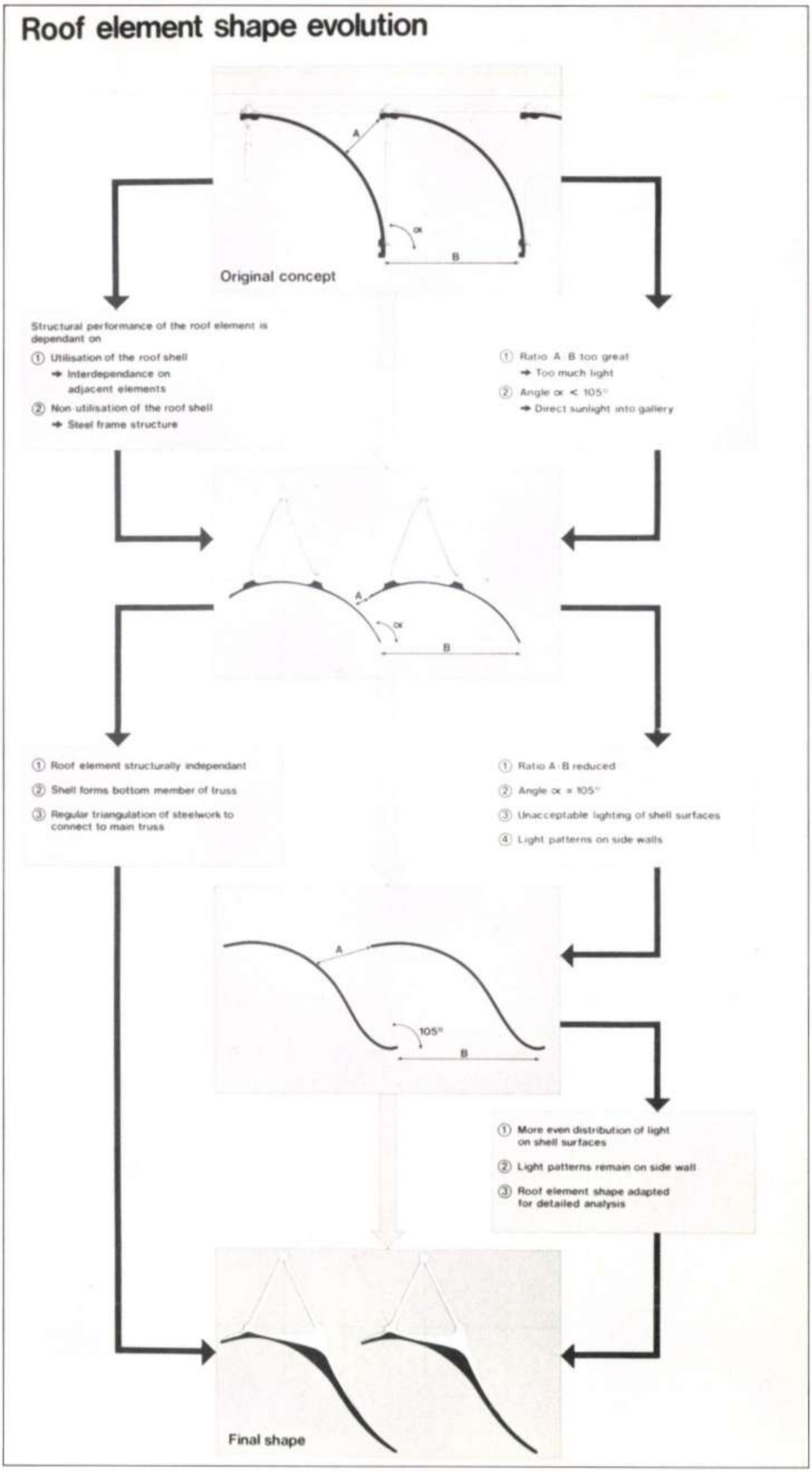
This shading function and the need to protect the spaces below from the wind and rain, and to perform as independent structural members, created the feel of a naturally occurring feature, like a tree. This led to the roof elements becoming known as the 'leaves'. Like the leaves of a tree, these elements should be very delicate, and precisely formed, containing no harsh lines or details, being organic in appearance. In addition, they should be capable of spanning the gallery spaces below, and supporting the roof glazing and any additional components.

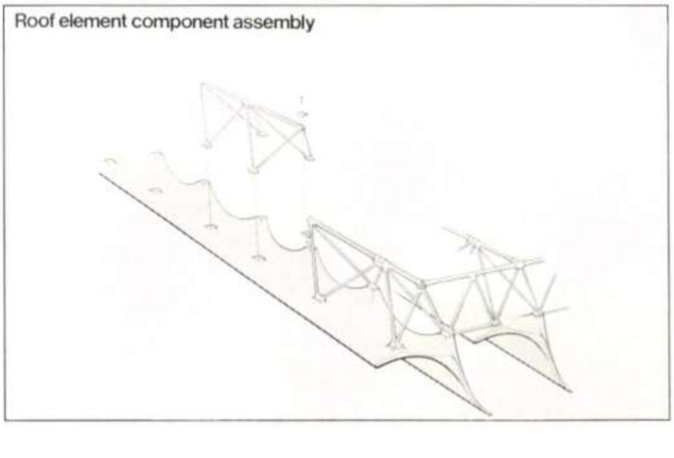
Roof materials

Ferrocement, a material traditionally used for boat building, and certain other thin wall, free-form structures, is ideally suited to this role. It consists of a number of fine steel meshes, impregnated with a cement-rich mortar, and exhibits properties sufficiently different from normal reinforced concrete, to be classified as a separate material. Acting compositely with the ferrocement leaf, to form a stable structural element, are a series of truss elements. As they are part of the leaf, they too need to be organic in appearance, and therefore, a material capable of being cast into various shapes is required. A particular type of cast iron, known as ductile iron or spheroidal graphite cast iron is to be used. The grade chosen exhibits tensile strength and elongation properties similar to mild steel.

Roof experimental development

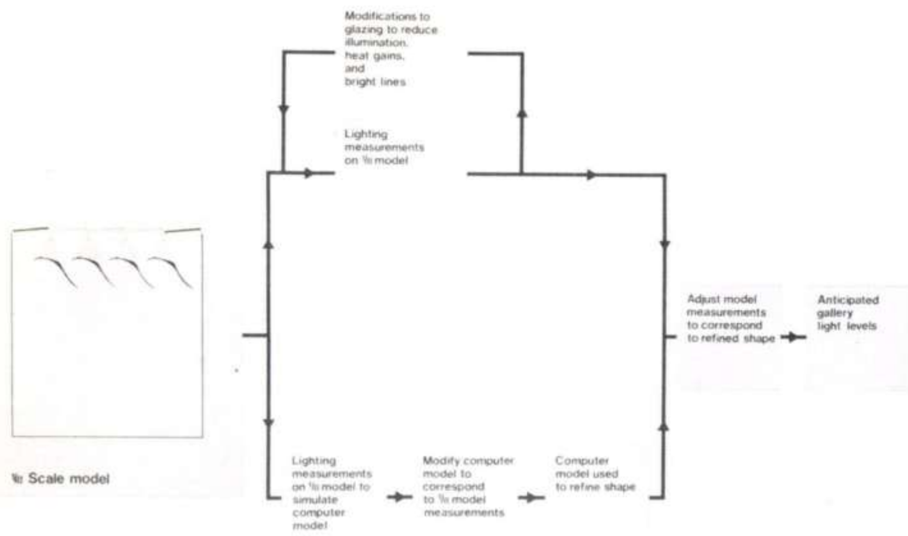
For the leaf to perform all the above functions satisfactorily, a period of research and development, including model testing, was undertaken to arrive at the shape of the leaf, from both lighting and structural aspects. From an understanding of the functions and performance, a rough idea of the proportions of the leaves was formulated. A crude model was made, based on these ideas, and the effects of the leaves on the internal lighting and appearance was investigated. Concurrently, a computer program was developed to predict the effect of the shape and reflectivity of the leaves on the internal light levels. Information fed back from these sources led to the design of a more refined leaf shape, which was incorporated in a second series of crude model tests. Following this, and a detailed analysis of this leaf as a structural element, a large-scale model was made. For this model, a glass was selected to minimize the transmission of heat into the gallery. This reduced the light





Figs. 6-10
Display panels produced for Houston presentation

Lighting analysis techniques



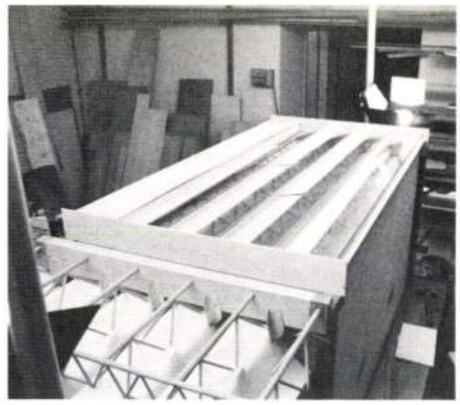
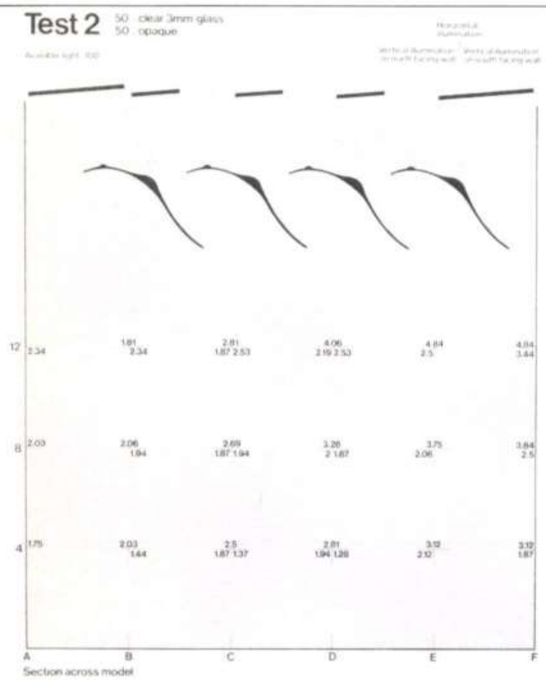
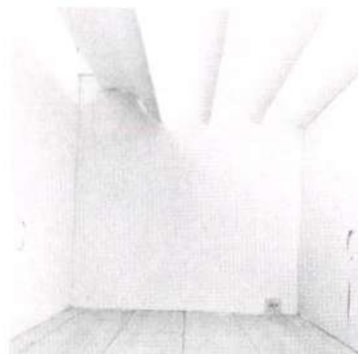


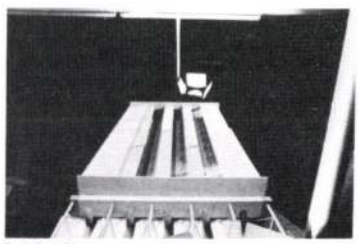
Fig. 11
The model used for light tests

Test 2



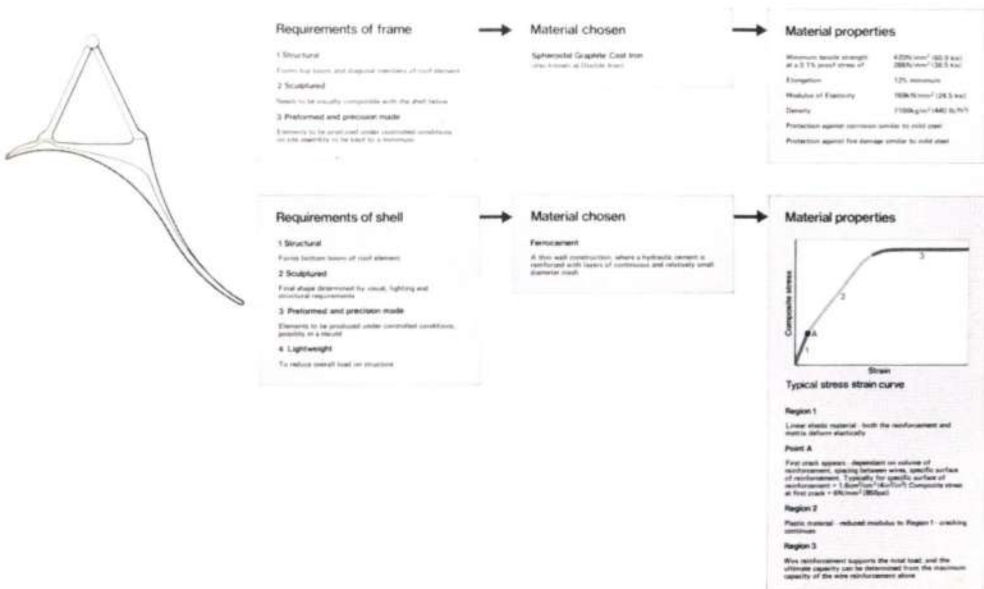


Interior of model



Roof of model

Roof element materials and properties



transmission, and, to maximize the levels of reflected light, it was decided that the surface of the leaves should be white. Light level readings, under both natural and artificial light, with various types of roof glazing, were made. These confirmed that the roof system was performing as predicted, and that the light levels were of the right order.

At the same time, models of the ductile iron truss elements were built. Being designed to support economically the structural loads imposed on them, a series of elements resulted, which also exhibited the organic appearance of the leaves.

Building design

The concept of the Treasure House is that of an environmental safe, designed specifically to achieve stable conditions, particularly of light, heat, and humidity. The original design was based on mass, using principally concrete for all walls and slabs. However, the construction of the walls has been changed recently to a dry form more commonly used in local construction practice. This will include dry cladding and lining panels, external insulation to smooth out external environmental changes, an inspectable vapour barrier to eliminate internal condensation, and an air-tight construction to prevent infiltration of external air. The structure will be a conventional steel frame, with in situ concrete being used for the floor and roof slabs.

The ground floor will include the public galleries, library, conservation laboratory, and registration facilities. The construction of the ground floor walls is dictated by the need to provide cladding which is similar to the balloon frame houses nearby. This will be formed of horizontal, overlapping wooden strips, surrounded by a frame made of steel channels. Because of the need for environmental control of the gallery spaces, these walls will also contain high levels of insulation, a vapour barrier, and a separate internal picture hanging wall. The steel supporting the platform roof above will also be contained within these walls. The ground slab will be of reinforced concrete.

The basement will include plant rooms, storage areas, photographic studios, and workshops. It will be constructed of in situ concrete, with special attention being paid to the waterproofing and level of the slabs due to the likelihood of flooding. Additionally, high risk plant, such as boilers and generators, will be located in a structurally isolated, blast-proof vault in the basement, designed to prevent a fire or explosion from affecting the remainder of the building.

The air-conditioning systems for each zone will be a minimum fresh air, constant volume, recirculation system. This has the advantage of reducing the risks associated with the intake of contaminated and polluted air into the building.

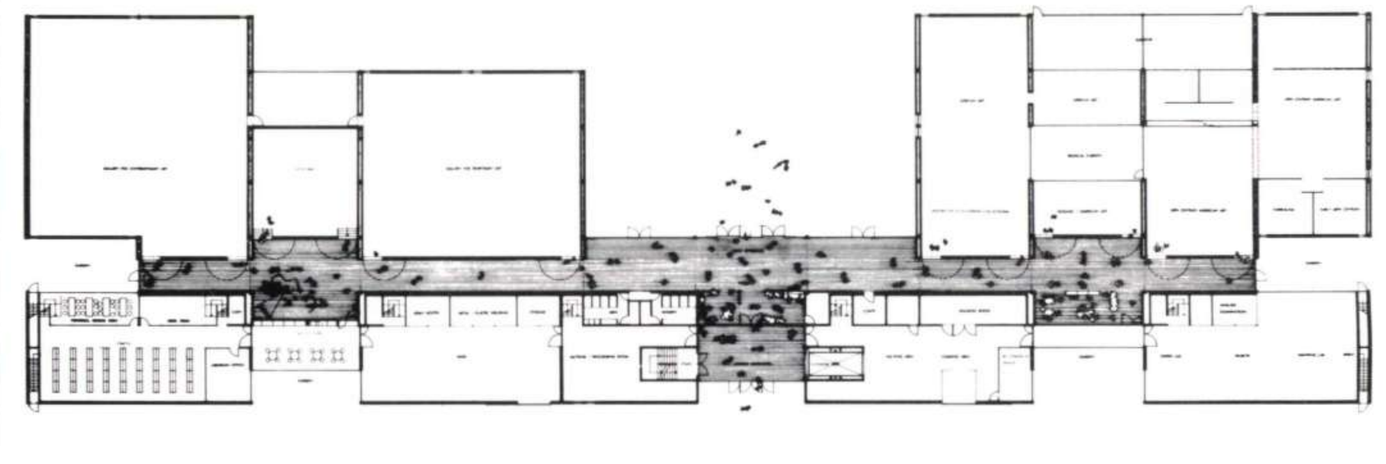


Fig. 12
Ground floor plan

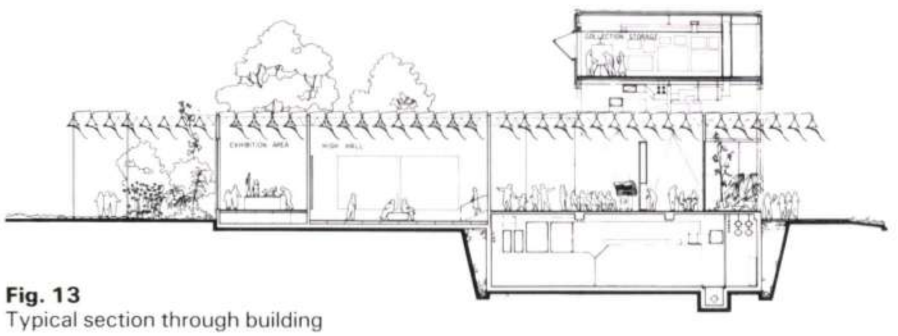


Fig. 13
Typical section through building

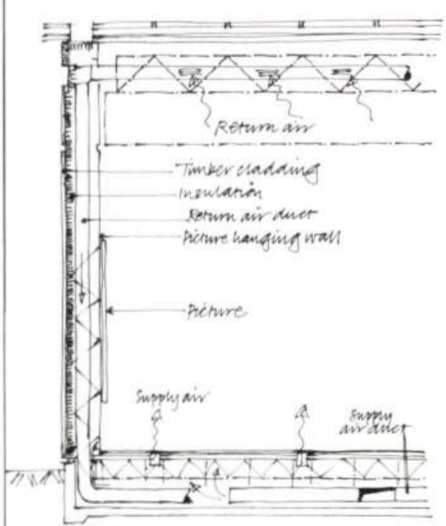


Fig. 14
Section through external wall

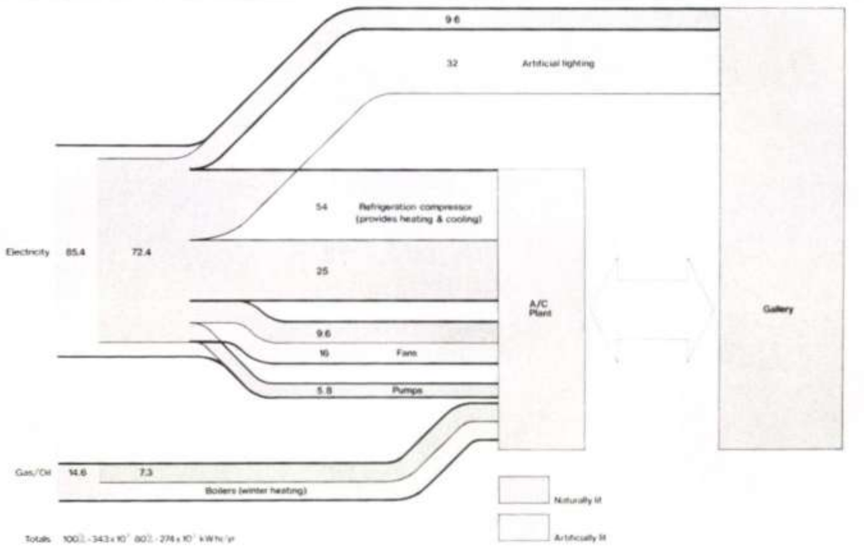


Fig. 15
Annual energy consumption
of an 80ft × 80ft gallery

These proposals and models formed part of the design development presentation to the client in Houston. Following this presentation, we were given the go-ahead for further development work on the platform roof.

Further development of roof design

Detailed development work proceeded on three fronts:

(1) Fabrication of the ferrocement leaf
(2) Fabrication of the ductile iron castings
(3) Construction of a full scale mock-up of a gallery to assess the lighting characteristics.

To assist in the practical methods of constructing the leaves, Windboats Marine of Wroxham, Norfolk, a boat-builder specializing in ferrocement boats, was used. A full-scale section of a leaf was made of polystyrene. This was then viewed and approved by the architect. A glass reinforced plastic mould was taken from the upper surface of the section and used as the master for all further development work on the leaf. At this stage, it was envisaged that the leaves would be fabricated by placing a cage of reinforcement and mesh into the mould, placing the mortar and finishing the inner surface by hand. A jig system for the bending and fixing of the mesh was devised and a number of cages for test leaves were built.

Two sections of leaf were made; one using the mould method and the other using the traditional hand placed method. Results of these were encouraging, but more development work was required. This is currently being carried out in Houston, where various ways of mass producing the leaves to the required quality are being investigated. Tests on various cements and sands are also being conducted to meet the requirements of reflectivity and surface texture.

To advise on the practical aspects of fabricating the ductile iron truss elements, Crown Foundry of Northampton was appointed. Full-scale wooden models of the truss elements were made from our detailed drawings, and assembled on the polystyrene section of the leaf for the architect's approval. Following competitive bids to the main contractor for the supply of all ductile iron elements for the project, this British foundry was appointed. An acceptable specification and testing procedure has been agreed, and prototype testing successfully completed. The quality and tolerance of the castings produced to date are good, due mainly to the process employed. The complete roof unit will be formed by bolting the individual ductile iron truss elements to

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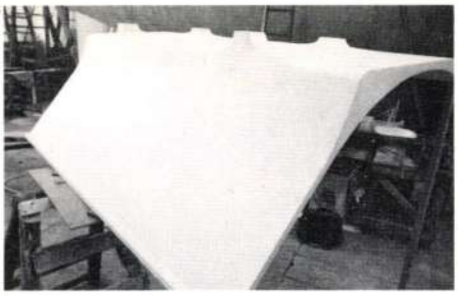
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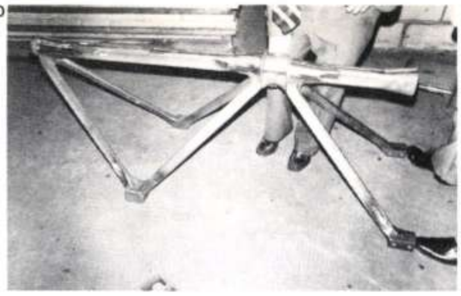
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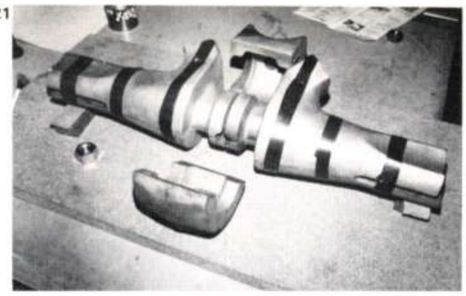
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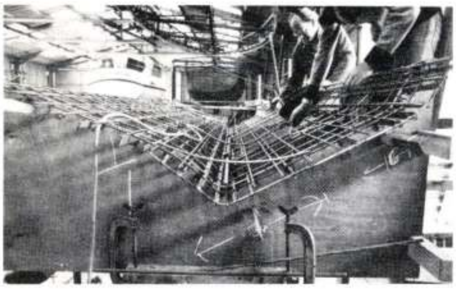
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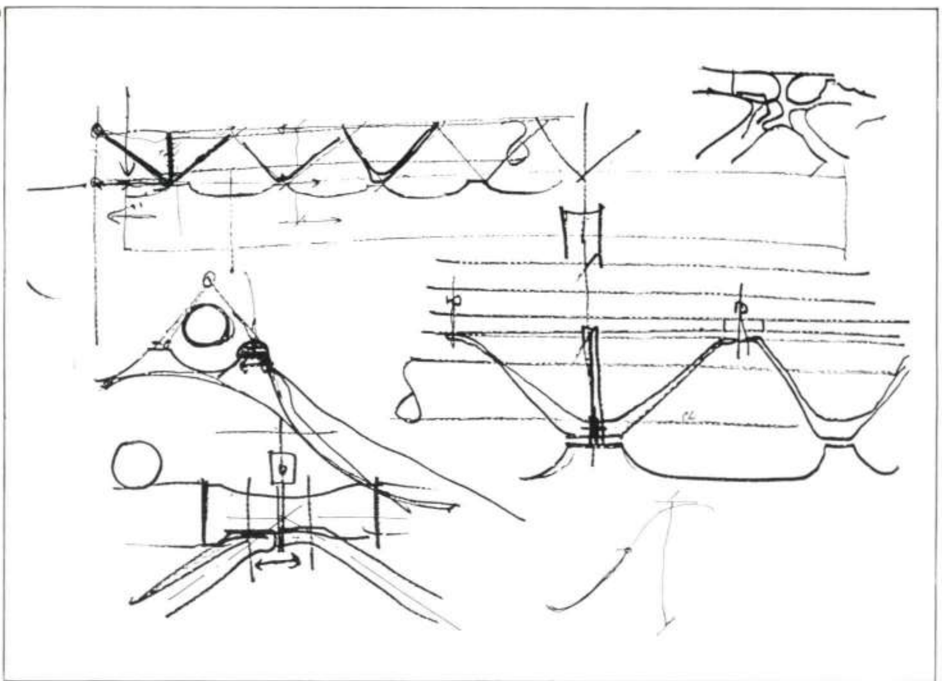


Fig 16
Full scale section of leaf

Fig. 17
Jig used for bending and fixing mesh and reinforcement

Fig. 18
Traditional ferrocement process—forcing the mortar through the mesh by hand

Fig. 19
Early Renzo Piano sketch of ductile iron details

Fig. 20
The first ductile iron elements—hastily assembled at the foundry

Fig. 21
Detail of connection of ductile iron elements

Figs. 22-23
Internal views of full-scale mock-up of gallery in Houston

Photos: Ove Arup & Partners except where otherwise stated

the ferrocement leaf, and the ductile iron elements themselves will be joined together using a clamping system utilizing a metal-filled epoxy resin. The first production elements have been dispatched to Houston to be used in a load test of the complete roof element.

At an early stage in the project the client decided on the necessity of building a full-size gallery mock-up on the site in Houston. This was principally to make an assessment of the appearance of the gallery and to judge the suitability of the lighting solution. It also provided an opportunity to take measurements of the lighting levels. General reactions to the mock-up were favourable although it did show up a few problems. The internal light levels were in excess of the design brief and there was some glare from the leaves due to the level of light transmitted through the roof glazing. In addition there was a problem with streaks of sunlight entering the gallery through the roof in the early morning and the late evening in midsummer. To solve these problems, roof glass with a lower light transmittance value was substituted and the gap between the leaves was temporarily adjusted. Based on the measurements taken in the mock-up, the annual and daily variation in light levels was computed, and a report, relating these to the target annual lux hours, was produced. In addition to the light tests, a very simple plenum floor air-conditioning system was built into the mock-up. This enabled us to measure the temperature stratification between floor and roof. The mock-up is currently in the process of being adjusted to take all modifications into account, for final approval by the client.

The construction drawings, specifications and further detailed design on the project have now been handed over to the Houston team of architects and engineers. We anticipate that construction will commence in the spring of 1983, subject to the approval of the project budget, the mock-up, and the ferrocement fabrication process.

Credits

Client:
The Menil Foundation, Houston, Texas

Architects:
Piano and Fitzgerald, Houston, Texas

Engineers:
Ove Arup & Partners, London
Galewsky & Johnson, Beaumont, Texas
Gentry, Haynes & Whaley, Houston, Texas

Main contractor:
E. G. Lowry Inc., Houston, Texas

Appraisal of existing ferrous metal structures: Part 2

John Blanchard
Michael Bussell
Allan Marsden

Edited by:
Poul Beckmann

Assessment of member strengths

A pre-requisite of the final assessment of member strength for future use or continued present use is that the structure has been surveyed in sufficient detail to establish the dimensions, the connections, and other details of the parts of the existing construction that are to remain. These, together with the estimated or measured material strengths and loads, form the data to be employed in the assessment.[4]

Two approaches are available: using past or present codes or other published guidance, or starting from first principles. The first is obviously quicker, simpler, and hence preferable where guidance exists and the answer it provides is good news: the second may be necessary where this is not the case. Experience has shown that it can be effectively used to justify columns.

Regardless of which approach is used, calculations should generally be made in terms of (unfactored) service loads and permissible stresses, in order to facilitate comparisons with published data.

Cast iron columns: published guidance

The most concise and useful guidance currently available is published by the Greater London Council.[2] This reproduces the 'basic' permissible stress figures from the 1909 London Building Act quoted earlier, and includes also tables and graphs showing the reduction of permissible stress due to column slenderness and end fixity condition.

This source appears to be generally acceptable to District Surveyors in central London as a basis for assessment. Such acceptance could perhaps be disc:eetly cited as precedent for agreement elsewhere.

Wrought iron and steel columns: published guidance

The 1909 London Building Act gives permissible stresses for varying slenderness ratios and end fixity conditions. These remained in force until the advent of *BS 449: 1937*, which in turn was superseded by the 1959 edition. *BS 449* applies to steelwork only (wrought iron being obsolete by 1937).

Assessment should be made using the stresses from these documents appropriate to the date of the structure. Where an enhanced permissible basic stress has been agreed by the building control authority following strength tests, it would be reasonable to factor up tabulated stresses correspondingly.

Beams

The 1909 London Building Act gives permissible stresses (quoted at the end of this paper) which may be used to assess cast and wrought iron, and steel pre-dating 1937. No reduction factors were given for slenderness; instead, it was required that beams be 'secured against buckling' if the span exceeded 30 times the width of the compression flange or the depth exceeded 60 times the web thickness.

Deflections were to be calculated and were required to be less than one four-hundredth of the span, unless the span-depth ratio was less than 24.

The 1937 edition of *BS 449* similarly required lateral restraint to compression flanges at a maximum spacing of 20 times the flange width; calculated deflections were not to exceed 1/325th of the span unless the span-depth ratio was less than 24.

The 1959 edition introduced the more familiar reduced permissible stress associated with slenderness.

Assuming that section sizes and material strengths are known together with the date of construction, it seems reasonable to assess existing elements and connections in accordance with the relevant published guidance, i.e. cast and wrought iron, and pre-1937 steel, in terms of the 1909 London Building Act; post-1937 steel to the then-current version of *BS 449*. For steel, a rational alternative is to derive the basic permissible tensile stress from test results as previously described and carry out the assessment calculations in accordance with *BS 449: 1959*, factoring the various permissible stresses in proportion to the basic tensile stress.

The approach should of course be agreed with the building control authority before time is expended on detailed justification.

Filler joist floors

The filler joist floor is a particular form of construction in which concrete, usually with a flat soffit and unreinforced, acts as a slab, spanning between steel or wrought iron joists at 0.5-2m centres. It evolved directly from the brick-arch 'fireproof' floor used in mills and warehouses, and would doubtless be more popular today if reinforced concrete were not so widely used.

It should be noted that the slabs of filler joist floors are often found to be of 'clinker' concrete – the aggregate being coal or coke clinker or breeze. This can, in damp conditions, cause severe chemical attack on the metal joists, and it may also become a matter for acceptance by the building control authority whether the floor can be regarded as non-combustible, when the aggregate contains a high proportion of unburnt coal (up to 50% has been found in one instance!).

The metal joists should be assessed as for beams. *BS 449: 1959* recognizes the enhancement of strength which is achieved by encasing the compression flange in concrete, and the section may be assessed on the basis of composite action where this is present. (A building control authority may, however, demand justification for the shear transfer at the concrete – steel interface. This can be difficult to substantiate, particularly where clinker or breeze concrete is present in older structures. A load test might then be the only positive proof.)

Assessment of column strengths from first principles

Empirical formulae

Around 1900 several empirical formulae for pinned-end columns were proposed and adapted for building codes, remaining in use until after the Second World War. Generally, these specified a limiting slenderness-ratio above which the failure load was taken as equal to the Euler critical load:

$$P_E = \pi^2 \frac{EA}{(L/r)^2}$$

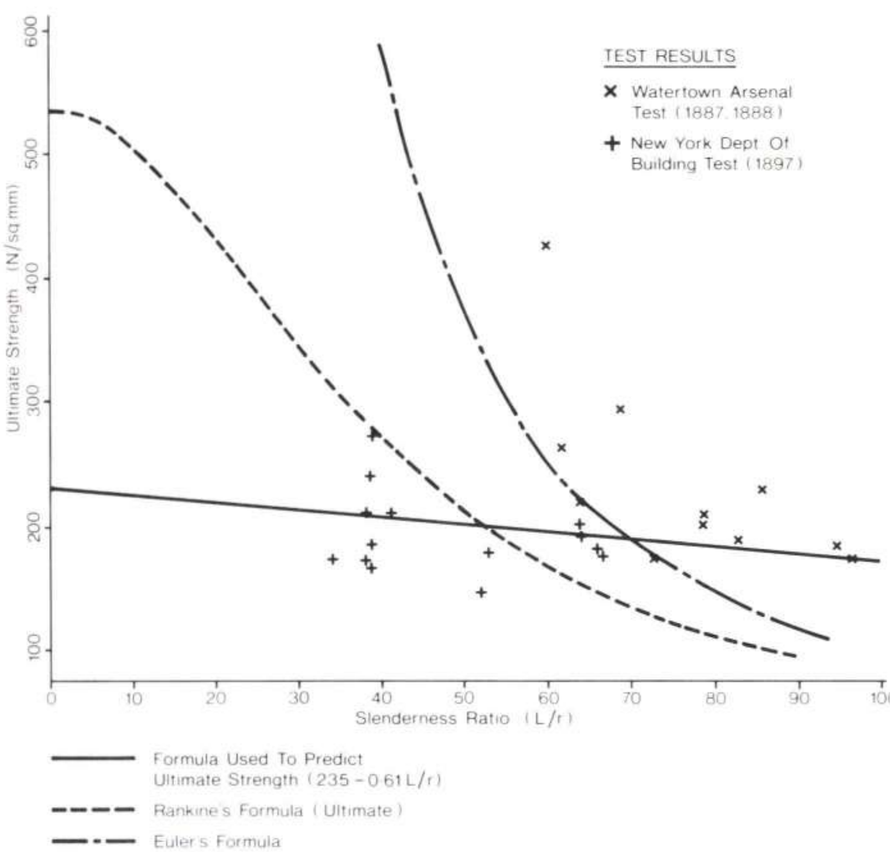


Fig. 4
The ultimate compressive strength of cast iron columns as predicted by formulae and found by tests in the USA.

Note the variability of test results, indicating the need for a generous factor of safety.[2]

For smaller slenderness ratios the failure load was given by some simple function of L/r.

These formulae were chosen to fit experimental results taken from tests in which the columns were specially straightened, the loading specially centred, and every precaution taken so as to attain as near as possible the 'ideal' column. These formulae are not very suitable for use in retrospective calculations. This is partly because their correct use would require knowledge of the material properties, not only of the column under investigation but also of those in the original tests. But a more important objection is that the choice of safety factor, difficult enough for a column, is made harder because it must take account of the imperfections in a practical column. The safety factor would then be expected to vary with the length of the column and with its slenderness ratio.

To get round the problem it seems logical to use basic formulae for the failure load which include specifically the effect of imperfections. This is a logic which has been adopted by most building codes since the 1930s, including *BS 449*.

However it is useful for comparison to recall the more important of the empirical formulae for the value P of the failure load.[5]

Rankine-Gordon (1858-1902)
$P = Aa/(1 + b(L/r)^2)$

where the recommended values for 'a' and 'b' were:

		Cast iron	Wrought iron	Mild steel
a	tons/in^2	36	16	21
	(N/mm^2)	(552)	(248)	(331)
b		1/1600	1/9000	1/7500

where A is the cross-sectional area

Tetmajer's Straight-Line Formulae (1896) for mild steel with $L/r > 105$:

$$P = A\,(304 - 1.118\,L/r)$$

Johnson's Parabolic Formula (1893)

$$P = A\,(a - b\,(L/r)^2)$$

where the recommended values for 'a' and 'b' were:

		Cast iron	Mild steel	Nickel steel
a	tons/in^2	27	18	25
	(N/mm^2)	(414)	(276)	(379)
b	tons/in^2	0.0028	0.0006	0.0011
	(N/mm^2)	(0.0431)	(0.0092)	(0.0172)
for L/r >		70	122	105

The Rankine-Gordon formula is reckoned to be best for cast iron and the Johnson formula is preferred for mild steel.

Calculation on the basis of assumed imperfections

The principal imperfections in an actual, not 'ideal', column are:

(a) accidental eccentric application of the load (i.e. other than design values of eccentricity determined, for instance, by the position of a beam bearing relative to the column centre line)

(b) non-homogeneity of the material, i.e. variation of the material properties especially across the cross-section.

(c) geometric inaccuracies in the cross-section

(d) initial curvature of the column.

All these imperfections can be represented by an initial end eccentricity of the load of amount e mm, together with an initial bow of amount c mm. In general the top and bottom end eccentricities may be different and then e is taken to be their average value (when calculating the total eccentricity at mid-height). For convenience the bow is assumed to have the shape of a half sine-wave (of amplitude c).

Suppose that the determinate end eccentricities (due to known beam positions) are also, after averaging, included in e and suppose that a vertical load P is applied with this eccentricity to a pin-ended column; then the bending moment due to the eccentricity of the load from the centroidal axis of the column will cause an additional deflection, which will have approximately the shape of a half sine-wave of amplitude

$$\left(\frac{4}{\pi}e + c\right)\alpha/(1-\alpha) \quad \text{where } \alpha = P/P_E.$$

(This formula is exact for the c term and approximate for the e term where it represents the first term of a Fourier expansion for a linearly varying eccentricity. The approximation gets better as P increases and this first term tends to predominate.)

The total eccentricity at mid-height will then be

$$\left(\frac{4}{\pi}e + c\right) \quad /(1-\alpha) - (4/\pi - 1)e$$

where the second term may be neglected.

The cross-section is then checked for the load P at this eccentricity. The load P at which the capacity of the cross-section is just not exceeded is taken to be the failure load of the column.

Assessing the assumed imperfections

What values are to be assigned for the initial imperfections e and c? Salmon (1921)[6] collected the available evidence (back to 1820) and suggested some values. The evidence was conflicting and confused and Salmon did not succeed in resolving the confusion. In the following, r is the radius of gyration and D the greatest overall dimension of the cross-section in the plane of bending.

(a) For accidental eccentricity of load application Salmon suggested a value for e of r/10 or, in a form which he thought more useful, L/1000. The fact that the same data was represented by two such very different expressions is a measure of the unreliability of his conclusions.

(b) To account for the non-homogeneity of the material, he gave a formula for the eccentricity in solid compact section which reduced to D/50 for circular sections and D/40 for rectangular sections. For built-up (flanged) sections he suggested D/20. He did not give any values for hollow circular sections but on his arguments D/30 would probably be appropriate. His values were based on rather theoretical grounds, probably applying to cast iron. It is not clear how or whether this effect was excluded from the estimates of e in (a) above. They take no account of the effect of residual stresses from the manufacturing process. It is worth noting here the great difference between the properties of cast iron at the periphery and at the core. Athough this would not normally influence the eccentricity, it affects, of course, decisions on the making and interpretation of tests on small specimens. Collet-Meygret and Desplaces (1854)[7] made experiments on cast iron bars such as were used for the Rhone Viaduct at Tarascon. They concluded, supposing the peripheral zone to be 5mm thick, that E varied between 12,000,000 kg/m^2. (120,000 N/mm^2) at the periphery to one quarter of that value at the core. This E was tensile and presumably the initial value. Similarly, the tensile strength in one piece varied from 40,000,000 kg(m^2. (400 N/mm^2) to one half of that value.

(c) To allow for geometric inaccuracies in the cross-section, Salmon suggested an eccentricity of D/80 for a flanged or built-up section. This was deduced theoretically from the then current specifications. He did not give a value for hollow circular sections and pointed out that, for cast iron, the position of the core was so variable that it would be unreasonable to do so. Although Hodgkinson[7] remarked that the displacement of the core 'does not produce so great a diminution in strength as might be expected, for the thinner part of a casting is much harder than the thicker, and this usually becomes the compressed side'. As a corollary he noted that 'To ornament a pillar it would not be prudent to plane it'. He considered that, for Low Moor Iron the crushing resistance at the core was three-quarters of that at the periphery. Salmon proposed that the total eccentricity for (b) and (c) should be assumed to occur half as a constant eccentricity e and half as an intitial curvature c.

(d) He studied a large number of test reports and concluded that a reasonable upper value for the initial curvature, c, of a practical column was L/750. Alternatively, he thought, for convenience this could be taken as 0.058 L/r (mm). This part of the study was specifically limited to wrought iron and steel. There was some suggestion that the initial curvature of cast iron columns would be rather greater.

Kayser (1930) and Timoshenko (1936)[8], leaning rather heavily on Salmon's work, proposed that all the imperfections could be allowed for by an initial curvature, c = L/400 without any end eccentricity e. Following Jasinsky (1908), Timoshenko also thought that for small slenderness ratios the imperfection would be better represented by an initial curvature S/10 + L/750. Here S is the core radius, that is $2r^2/D$. S has the value D/8 for solid circle, D/6 for a solid rectangle, D/4 for a hollow circle and approaches D/2 for a flanged section. Both these proposals of Timoshenko seem to be intended for wrought iron and steel rather than cast iron.

Implicit in the strut formula of *BS449: 1959* is an initial curvature, $c = \frac{0.6}{D}\,L/100)^2$, again without any end eccentricity, e.

This has the notable advantage that the calculated failure load will depend only on the slenderness ratio (L/r) and is independent of the shape of the section. It seems sensible that the bow/height ratio should increase with L/r; of two columns of the same height the thinner would be expected to have the larger initial curvature. The *BS449* requirement is more onerous than Timoshenko's L/400 if L/D is greater than 42, (i.e. if L/r is greater than 170 for solid circles, 145 for solid rectangles, 120 for hollow circles and 85 for flanged sections). *BS449: 1937* implied an initial curvature of $\frac{0.6r}{D}\left(\frac{L}{100}\right)$. This is more severe than the later assumption when L/r is less than 100.

It will be seen from all the above that the choice of values for initial imperfections is an act of faith rather then science. Any information that can reasonably be obtained by in situ measurements should be gratefully received. It seems feasible to measure the initial curvature by plumbing. But it must be remembered that, if the column is loaded at that time, a back-calculation must be made to deduce the unloaded initial curvature c. And this value must then be increased by terms representing the imperfections of type (a), (b) and (c) that cannot be directly measured.

For hollow cast iron columns it will be essential to measure the eccentricity of the core of the column. This can easily be done by measuring the thickness at points round the circumference. If this is done at top, bottom and mid-height of each column, it should give enough information to decide on end eccentricities and bow. Some safe adjustment would be made for the effect of peripheral hardening and the values must be increased by terms representing imperfections of types (a), (b) and (d).

Failure loads of wrought iron and mild steel columns

With values assigned for the initial imperfections, including any determinate end eccentricities, and knowledge of the stress-strain properties, the calculation of the failure load is straightforward for materials with a well-defined yield point such as these. The failure criterion at the critical cross-section is taken to be the first occurrence of the yield stress. It is then reasonable to assume, for the calculation of P_E, the values of E as usually defined. Designing for the final total eccentricity given earlier leads to a quadratic equation for the failure load.

It is worth noting that, using *BS449*, the determinate end eccentricities are not included in the calculation of the additional deflection due to axial load. That is, the deflections due to applied end moments are not amplified by the ratio $1/(1-\alpha)$. This is usually excused on the grounds that the effect is small. This may be reasonable for normal building columns but is far from true for slender struts in transmission towers and similar structures.

Failure loads of cast iron columns

Having ascertained the initial imperfections and determined the material properties, merely brings us to the start of the problem for a material such as cast iron, with compressive strain-softening characteristics as shown by the stress-strain diagrams. For suppose that attainment of the rupture stress is taken as the failure criterion, how do we find P_E to calculate the final eccentricity? If we used the secant modulus to the rupture stress this would be safe but unreasonably so.

It will be recalled that the use of P_E is really only a convenient device to find the deflected shape due to the bending moments caused by the eccentricity of the load from the same deflected shape. So what is required to calculate P_E is an E-value which represents an appropriately weighted average of the secant modulus of every piece of material in the column. Given a stress-strain diagram it would be feasible to write a computer program that carried out such a trial and error process (trial and error because the final deflected shape is, at first, unknown). It would be feasible but very formidable and completely unjustified by the quality of our input information. In particular, it was blithely supposed above that the engineer was given a stress-strain diagram. Such information would hardly come as a gift and, in practice, it is unlikely that enough material would be available to determine representative stress-strain relationships for each type of column. Substantial pieces of metal would be needed to obtain the properties in the correct direction.

Some improvement on the simple approach first described can be obtained by using a limiting stress smaller than the rupture stress. At some optimum value the loss of calculated capacity due to the smaller allowable stress will be just balanced by the increase of the secant modulus. For this sort of calculation, or for that described in the next paragraph, it would be appropriate to use a typical stress-strain curve taken from the literature rather than one obtained by expensive tests.

However, this curve must be correlated with the actual material and simply measuring the crushing strength will not suffice for this. It would also be necessary to measure the stress for a given strain, say 0.3%, and, probably, to determine the initial E.

A greater improvement can be obtained, at the expense of some labour, by treating the critical cross-section as made of two materials, one on each side of the centroidal plane. This corresponds to the problem of bending of a beam with different modulus in tension and compression. Weighted average secant moduli for each half of the cross-section can be chosen, say E_1 and E_2. Then the curvature of the whole cross-section can be deduced from a single, reduced, modulus E_r given, for solid rectangles, by

$$E_r = \frac{4\,E_1\;\;E_2}{(\sqrt{E_1}\;+\sqrt{E_2})^2}$$

For other cross-sections, similar formulae can be deduced from the first principles that plane sections remain plane and that the internal stresses are in equilibrium with the applied eccentric load. This value of E_r can then be increased to allow for the less extreme stresses away from the critical section and the resulting value used to calculate P_E.

With cast iron there is the further complication that, for slender columns, the tensile stress may become critical before the compressive, and this must be checked. No doubt this is why building codes tend to limit the slenderness ratio of cast-iron columns rather severely. In addition, allowance must be made, for hollow columns, of the effect of any displacement of the core on both the stiffness and resistance of any cross-section.

Other end conditions

All the above discussion has been in the context of columns pinned at both ends. For other end conditions the column would be treated as pin-ended of length equal to the effective length estimated in the traditional way. In that way the assumed imperfections are those appropriate to the effective length and that seems sensible. With cast iron the non-linear stress-strain curve would make any pseudo-scientific calculation of the effective length peculiarly difficult.

Pre-war literature often refers to an end-condition described as flat-ended. This seems to be a rather academic distinction arising from the fact that test specimens often had their ends pointed in the usual pursuit of the 'ideal' column. By comparison, columns with top and base plates could develop significant end moment before the bearing surfaces started to open up. In the 19th century the effective length for a flat-ended column was often taken as that for a column fully fixed at both ends. Later it was given a value closer to that for a pinned-end column. The slightest rotation of the horizontal structure would invalidate the former assumption even if the end plates did not separate.

Safety factors

Suppose a column failure load based on assumed initial imperfections has been found. A safety factor has now to be chosen to convert this to a safe working load. Apart from accounting for variations in the applied load this should allow for any unseen weaknesses in the column or in the calculations and for the effect of residual stresses. Its value will depend on the initial imperfections and on the material properties; in fact, it will depend on whether a partial safety factor has been incorporated in the estimates of these.

Salmon (1921) thought that the safety factor should be 20 or 25% greater than for a comparable tensile member, on the grounds that the strength of a strut is far more sensitive to local flaws and weaknesses. For mild steel the safety factors given explicitly by *BS 449* are available as a yardstick. These were 2.34 in 1937 and 1.7 in 1959.

For cast iron, Salmon reckoned that the safety factor should be twice that for steel but this was in the context of safety factors on the empirical strength formulae. Some greater margin than for steel would be appropriate because of the ever-present hazard of hidden flaws in the casting; but the margin would depend on how conservative the method of calculating P_E and the failure load had been. No yardstick can be obtained from contemporary codes because the assumed eccentricity method was never used for cast iron. Salmon thought the use of the method was not advisable.

The engineer may wish to compare his safe working loads so obtained with values deduced from the empirical formulae. Indeed for cast iron this would be an imperative rather than an option. For this calculation he will need a larger safety factor which now takes account of the imperfections in a practical column compared with the experimenter's ideal column. This safety factor will also take into account the fact that, however well he knows the properties of his own material, he knows nothing of the properties of the original experimental material.

To quote Salmon[6] (1921) yet again, (but he did devote much effort to the problem of the column), he thought that an appropriate value for steel or wrought iron was 4 + L/20D. For cast iron he thought this factor should be doubled and should certainly never be less than 5. As far as can be judged, contemporary code-makers were less pessimistic than Salmon. By the 1930s codes for mild steel columns were using safety factors of about 3. The 1909 LCC rules for cast iron columns implied safety factors of between 7 and 8 for L/r less than about 70. The 1937 German rules for cast iron (DIN 1051) catered, one supposes, for a more sophisticated and controlled material. The safety factor in these, for L/r less than 80, was about $4\frac{1}{2}$ on the Johnson parabolic formula. For more slender columns the safety factor was exactly 6 on the Euler critical load, P_E, calculated using an E of 1,000,000 kg/cm^2 (98,000 N/mm^2). (When calculating P_E, if required for the empirical formulae, it is reasonable to use the initial value of E since the column is lightly stressed under the Euler load. The value adopted should also give continuity at the transition between the two parts of the formula.)

Conclusions

Old mild steel and wrought iron columns should be analyzed on the basis of assumed imperfections with the empirical formulae used only as a 'long-stop' check. The calculation will differ little from one in accordance with *BS 449* using a measured sample yield strength. Enough measurement should be made of visible imperfections, such as initial curvature, to determine whether the construction is exceptionally imperfect (or perfect). Where measurement imperfections are used as a basis for calculation they must be augmented to allow for invisible imperfections. The resulting design eccentricity should be handled with care if it differs from those implied by *BS 449* (1937 or 1959).

The safety factor chosen will depend on the era of the construction and a judgement on the degree of control exercised in the work. It will often be two or less for mild steel. A check for wrought iron columns would tend

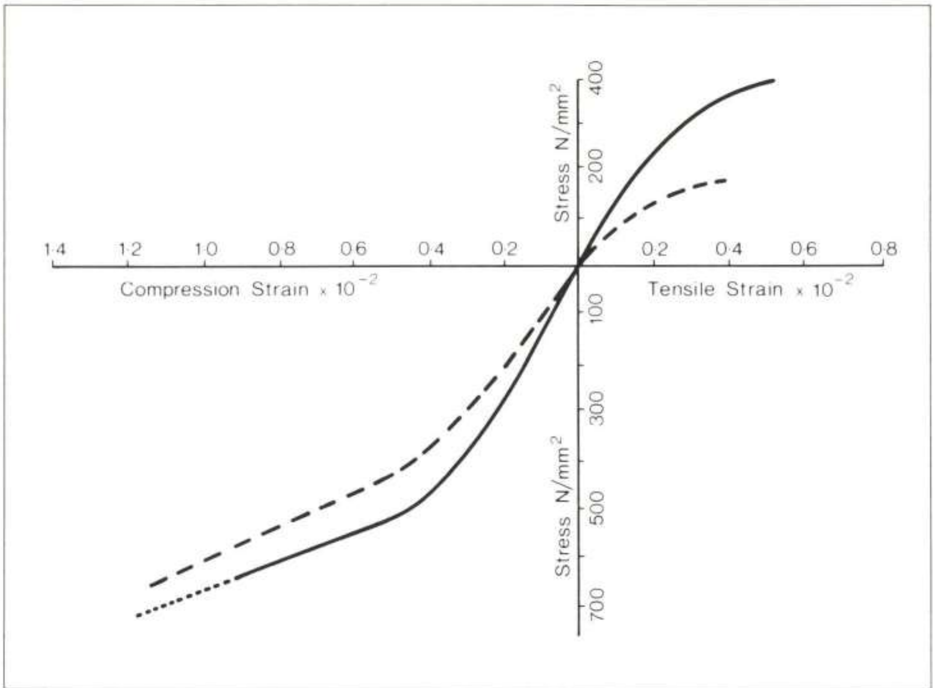
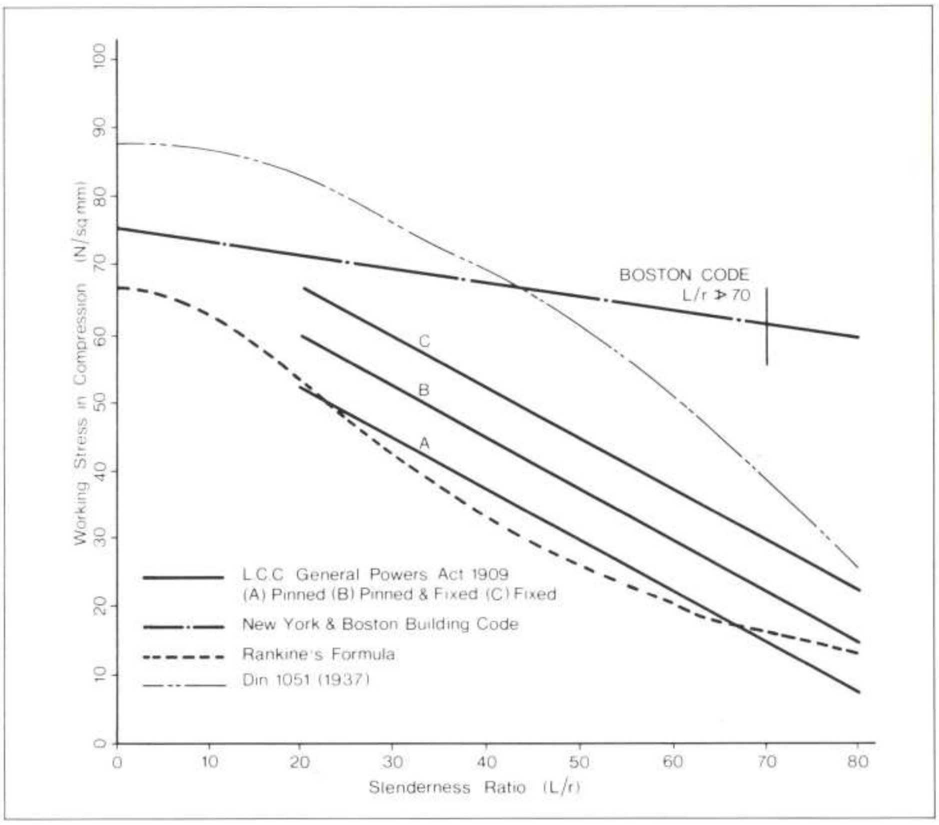


Fig. 5
Stress/strain curves for typical cast irons used structurally. Note the non-linearity, and the relative weakness in tension.[3]

Fig. 6
Permitted working stresses in compression for cast iron columns as defined for London 1909 (lines A,B & C). Other lines are for comparison of Rankine's formula and contemporary US/German practice.[2]



London Building Act (1909)
Permissible working stresses

	Tons/in.²	(N/mm²)
Cast iron		
Tension	1.5	23
Compression	8.0	124
Shearing	1.5	23
Bearing	10.0	154
Wrought iron		
Tension	5.0	77
Compression	5.0	77
Shearing	4.0	62
Bearing	7.0	108
Mild steel		
Tension	7.5	116
Compression	7.5	116
Shearing	5.5	85
Bearing	11.0	170

BS 449, 1937
Permissible working stresses for mild steel

	Tons/in.²	(N/mm²)
Tension	8.0	124
Compression	8.0	124
Shearing	5.0	77
Bearing	12.0	185

to be less conservative since less is known about their behaviour, although there is no indication that this differs significantly from that of mild steel columns.

The analysis of cast iron columns on the basis of assumed imperfections is feasible. However, the non-linearity and greater variability of cast iron, in comparison with steel, will lead to a more extensive programme of sampling and testing. In addition, more reliance has to be placed on the site measurement of geometric inaccuracies and initial curvatures. This may not be a trivial matter; consider the difficulty of interpreting offsets from a plumb-line when the plane of worst curvature is unknown and it cannot be assumed that the column is cylindrical (it may taper or suffer from entasis). The answers obtained from all this work and some fairly complex calculation have then to be divided by a substantial and speculative safety factor to cover the possibility of hidden flaws. The engineer may then wonder whether his journey was really necessary. But if he relies more on the empirical formulae he meets the difficulty that, to paraphrase Col. Wilmot's remark in a report to the War Office (1858), 'To say that a column is cast iron conveys the same amount of information as saying that it is made of wood.' Any information that the engineer has obtained on the material and construction of the column cannot be incorporated directly in the empirical formulae; it can only be taken into account when assessing the safety factor.

No doubt the engineer will devise a compromise method using both approaches and choosing a testing/calculation plan in which the effort involved is commensurate with the quality and relevance of the answer obtained. But he should first reconsider whether he would not be just as well off using published permissible stresses such as those of the 1909 LCC Act.

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Two theatres in Scotland at Pitlochry and Dundee

Derek Blackwood
Charles Moodie
Eddie O'Donnell
John Holt

Introduction

The construction of a new theatre in Scotland is a fairly infrequent occurrence so we count ourselves very fortunate to have been members of the design teams for the two most recently completed.

Although of roughly similar seating capacity, the detailed descriptions below show that the design problems presented by the different briefs and sites have resulted in two very different buildings. In particular the sites have had a strong influence on the architecture. At Pitlochry the spacious rural setting allowed the building to spread out comfortably, while, in complete contrast, at Dundee the building had to be shoe-horned into the corner of a city square. Not surprisingly, the atmosphere in each theatre is quite different. At Pitlochry the foyer spaces are outward-looking, taking full advantage of the views over river and mountains, inviting a stroll in the fresh highland air during the intervals, while those in Dundee are cosy and intimate, appropriate to a small city theatre.

Further comparison would be invidious: each theatre is highly successful in its own environment and both are very welcome as the first permanent homes for two vigorous theatrical companies.

Pitlochry Festival Theatre

Architects: Law & Dunbar Nasmith

For 30 years there has been a Festival Theatre in Pitlochry, a small holiday town on the fringe of the Scottish Highlands surrounded by beautiful scenery.

Originally a large marquee served to house Scotland's 'Theatre in the Hills', but within two years, encouraged by the success of the enterprise and the fact that the marquee blew down, the company erected a steel-framed, asbestos-cement clad structure, thereby achieving an air of permanence. As the years went by, however, it became clear that the theatre society was there to stay but the building was not. The ageing, unsafe structure had to be replaced and a design team was appointed in 1967, led by the architects, Law & Dunbar-Nasmith and

Fig. 1
Pitlochry Festival Theatre: View from the north west. Foyers and restaurant at upper level overhanging dressing rooms

Fig. 2
Interior of foyer showing bar upper gallery and tented form of ceiling

Fig. 3
Auditorium
(All photos in this section: A. L. Hunter Photography, Edinburgh)

including ourselves as structural engineers to design a new building on a new site.

The long search for a suitable site close to the town centre then began and was thought to have been solved when in 1971 a likely site was found on the north bank of the River Tummel. A fine design for the briefed 700-seat theatre was produced but the effects of rampant inflation were now apparent and the costs exceeded the amount which the Board of Governors felt they could raise.

An interesting feature of this site was that, it being located on the river flood plain we had to carry out a flood study. Although situated downstream from the Pitlochry dam which is part of the Tummel-Garry hydroelectric scheme comprising nine interlinked dams and power stations, complete regulation of the discharge from the 1800 km^2 catchment area could not be guaranteed by the North of Scotland Hydro Board. This was borne out by the records of river levels available for the 25 years since construction of the dam and, by a stroke of good fortune, recorded at a point immediately opposite our site. The floor level was to be set above the predicted 100-year flood level but plant in the basements was designed to continue functioning underwater. Floods greater than the design flood or – the ultimate disaster – failure of the dam itself, were considered matters not for design but for insurance and discussion with a major company indicated that this could be obtained at an acceptable premium. However our flood predictions were not to be put to the test; this site could not, after all, be acquired.

The long search continued and in 1976, an eminently suitable site on the south side of the river, which would allow the new theatre to be positioned on higher ground, was discovered by the Society. By now, in deference to the grim financial situation, the brief had been slimmed down; it was to provide the simplest permanent replacement for the existing theatre to allow the company to continue the operation carried on successfully for the previous 25 years.

The new and final site was, in many ways, the best. Although on the south bank of the river and therefore farthest from the town it was still a relatively short walk via the old suspension bridge nearby and there was space for a substantial car park. The higher elevation of the building possible on this site, in addition to ensuring dry feet for the audience, opened up splendid views of the mountains rising to the north over the river and town.

Following the acquisition of this site, a new scheme was evolved in 1977 to meet the limited brief and the serious business of fundraising began. This operation was extremely successful, the largest single contribution being almost £0.5m from the European Regional Development Fund, the first grant of its kind in the UK. As a result it was possible to award a £1.3m contract for the building in September 1978. Prior to this, in June, the contractor had been given a letter of intent to allow pre-planning and programming to ensure a good start.

Design

The final design is for a single tier, fan-shaped auditorium of 540 seats surrounded by foyers, kitchen and restaurant (very much a feature of the old theatre). At the rear the stage is flanked by two large scene stores. One of these, separated from the stage by a double acoustic wall, is destined in a future phase to become a studio theatre for an audience of 160.

The architects have used the sloping site to advantage by locating the stage and scene stores at the south side, dug into the hill. This ensures that the entrance, the foyer and the crossover in the centre of the auditorium are all at ground level. The coninuing slope of the site to the north allows the introduction of toilets and dressing rooms below the foyer. This unusual, but practical, arrangement incidentally affords the actors the same splendid view over the river as the theatre-goers above.

The old theatre obviously did not have a fly tower, nor does the new one. Desirable as this would have been it was considered too expensive since it would have involved, in addition to the higher and heavier structure, the provision of a safety curtain and a sprinkler system—both unnecessary in the theatre as built, since stage and auditorium are treated as one space by the Fire Officer. However, a grid (of inverted cold-formed steel channels) is provided over the stage with overhead pulleys and handlines, allowing the support and flying of low sections of scenery.

The foyer spaces merging into the restaurant are dominated by the north-facing glazed wall (5m high in the centre section) cantilevered out from the dressing rooms below. The tent-like structure of the foyer deliberately recalls something of the old theatre's atmosphere but the two 7m high 'tent poles' are now of 200mm tubular steel and support a reinforced concrete plantroom overhead in addition to the steel-framed roof.

The auditorium is heated and ventilated by four heat pump units located on the roofs. As a result the amount of large trunking commonly associated with theatre buildings is significantly reduced. Other areas are heated with underfloor electrical cables augmented, in certain public areas, with turbo heaters. In the absence of a gas supply on the south side of the River Tummel the kitchen is provided with its own propane gas supply.

Within the auditorium there are two lighting bridges and six side lighting ports. Access ways are provided within the roof structure linking the control suite and the grid. Much of the lighting and sound equipment was new but, where possible, items from the old theatre were re-used.

Building materials

Three main building materials are used: reinforced concrete, brick and steel. The tanked, reinforced concrete sub-structure rests on rock and compact sands and gravels which were happily found to occur on the proposed location of the building. Elsewhere on the 1.8 ha site, ground conditions were not so favourable with peat and soft clays occurring. The main stabilizing elements of the superstructure are the 325mm thick loadbearing brick walls enclosing the auditorium. Against these walls (which are restrained at the upper level by the auditorium

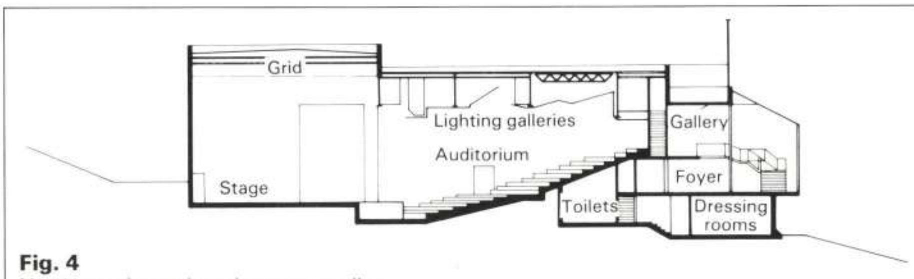


Fig. 4
North-south section along centre line

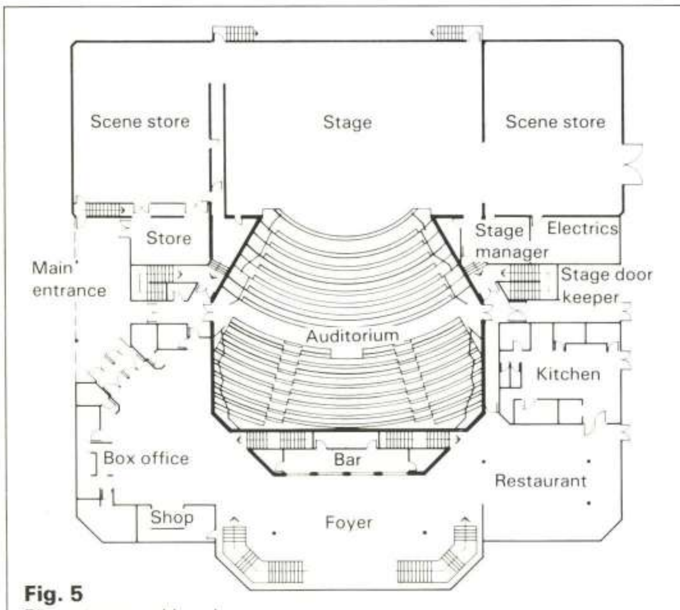


Fig. 5
Plan at ground level

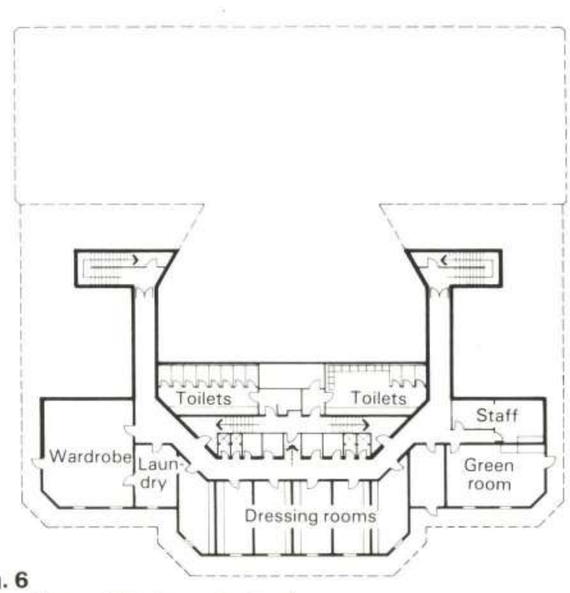


Fig. 6
Plan at lower dressing room level

roof steelwork) the foyer and restaurant structures are built in a 'lean-to' manner. The brickwork of the 12m high rear wall of the stage is stiffened by steel columns built into it.

Steelwork is featured architecturally in the foyer and restaurant areas where the main stairs to the upper gallery are constructed of tubular sections and steel plate and the glazed walls incorporate steel tubes and I sections. Elsewhere structural steelwork is used in the roofs but hidden from view by suspended ceilings. Although of conventional design and fabrication the steelwork is surprisingly complicated, for functional and aesthetic reasons, with very little repetition and this resulted in a high cost per tonne.

Work began in October 1978, unfortunately coinciding with exceptionally bad weather. Subsoil, which had appeared reasonable when examined in the summer conditions prevailing during the site investigation was reduced to a soft uncompactible mass. 50% more rain than normal fell during the period of the bulk excavation and, as a result, material which would have been re-usable in more clement conditions had to be taken off site and replaced by imported fill. However these setbacks only seriously affected the access roads and car park; the excavations for the theatre building which is founded on gravel and rock were unaffected.

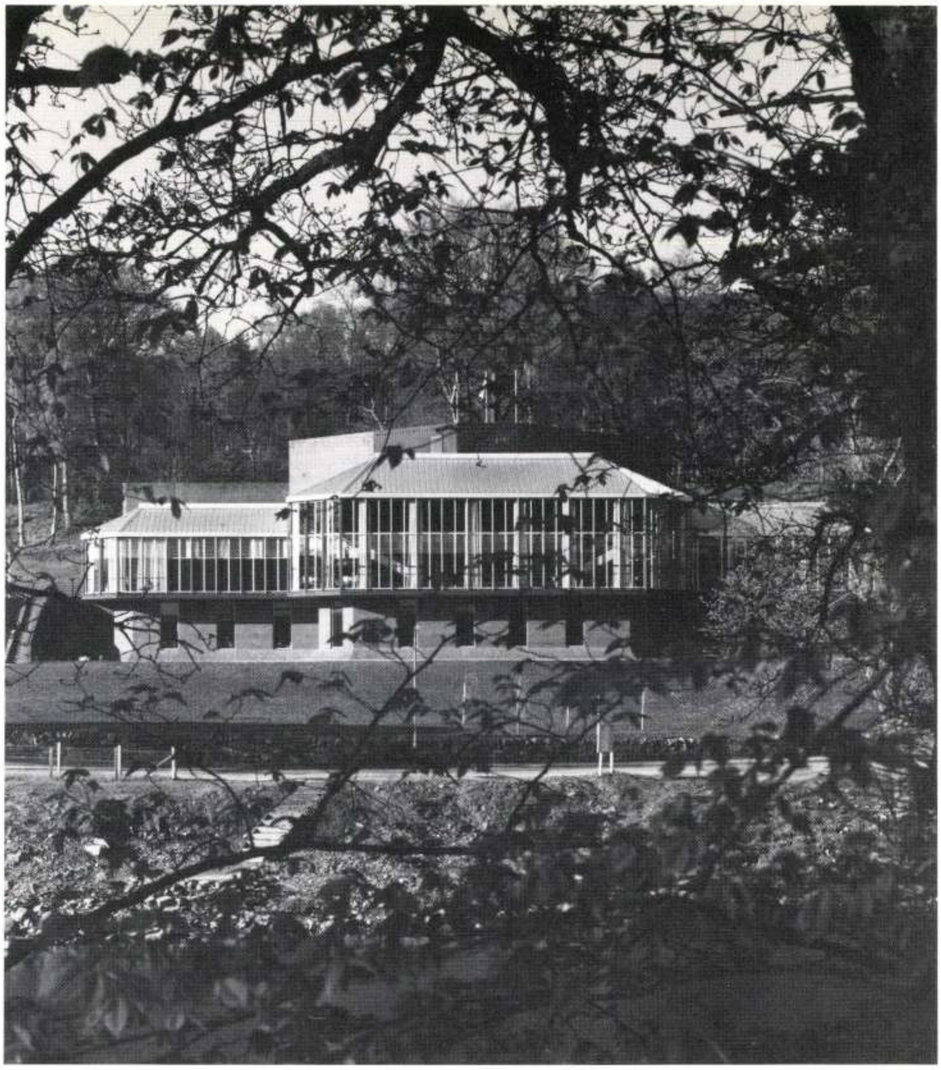
Inevitably the contractor experienced problems with the supply of materials, and also with obtaining sufficient labour, since the simultaneous construction of the Pitlochry bypass strained local resources and labour had to be brought in on a daily basis from towns 40 miles away. However, all the problems were finally overcome and the new theatre opened, as planned, on 19 May 1981, to a gala performance of Bridie's *Storm in a Teacup* exactly 30 years after the company's first performance.

The company at last have a permanent base that will enable them to fulfil, for many years to come, their promise to Pitlochry's summer visitors: 'Stay six days and see six plays'.

Fig. 7
Interior of scene store (Below)

Fig. 8
North elevation of theatre overlooking the River Tummel

Fig. 9
Foyer interior showing tended form of roof and fully glazed north wall



Dundee Repertory Theatre

Architect: Nicoll Russell, Architects Studio

The building of the new Dundee Repertory Theatre has proved the exception to two proverbs. Firstly, it has been possible to make a silk purse out of a sow's ear in that the fully equipped 450-seat theatre cost just over £1m and secondly, a quart has been put into a pint pot by providing such a theatre on a small plot of ground previously used for parking about 12 cars.

Dundee Repertory Company staged performances in a conventional theatre from 1939 until 1963 when the building was destroyed by fire. Thereafter, plays were produced in a small church converted to a theatre. This was always considered to be a temporary home for the company who were constantly searching for a site and funds to build a proper theatre.

In 1970 we were appointed as structural consultants for a project combining a new hall for the University of Dundee with a new Repertory Theatre. Shortly after the project started, the theatre was removed from the brief and only the hall was built. In 1974 we were appointed as consulting engineers (structural, mechanical and electrical) for a theatre sited on what remained of the hall and theatre site. A scheme was proposed, rejected on costs, the project given to another architect and eventually returned to the original design team in 1976. The project was to be funded by the Repertory Company, Dundee District Council, Tayside Regional Council, the Scottish Arts Council and the Scottish Tourist Board.

The design stage commenced early in 1977. Work on site commenced early in 1979. The first performance was in April 1982.

The theatre sits in a corner of Tay Square, an area in the centre of the city affording a few trees and car parking for approximately 50 cars. The architects for the project, Nicoll Russell, were conscious of the need to take every opportunity to conceal the fact that space was extremely limited and created a glass façade at the theatre entrance thus fusing the foyer with Tay Square. The glazed wall is over 7m in height extending from the ground to auditorium entrance level. In order to make the glazed walls as unobtrusive as possible there is little visible support to the glass which consists mainly of large panels and a folded plate form in plan with glass to glass jointing using adhesives.

Fig. 10
Main entrance (Photo: Spanphoto, Dundee)

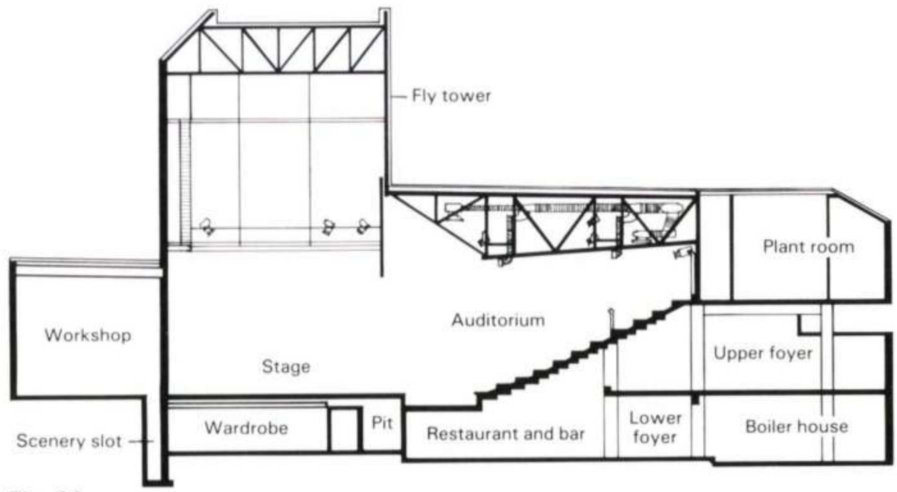


Fig. 11
Section on centre line

Fig. 12
Ground floor level

Circulation

Immediately behind the glass façade is a small entrance foyer with a box office and access to the theatre administrative offices which flank the stage. From the foyer one may proceed to the auditorium via a cantilevered reinforced concrete staircase which curves just inside the glass façade up to the upper foyer. The entrance foyer also gives access to a bar and restaurant nestling below the stepped reinforced concrete auditorium slab. Lighting to this area is by theatrical type spotlights directed along the concrete and blockwork surfaces. The intention was to create an area at the entrance which could be changed by displays to echo whatever production is on.

The restaurant and bar are open for business every day and not only during performances. It is hoped that in this way the theatre may provide a day-to-day meeting place close to the city centre.

The upper foyer is immediately above the lower foyer and does not infringe on the restaurant, making it possible to look from the upper foyer into the restaurant. The edge of the upper foyer is protected not by heavy balustrading but by thin vertical steel rods to create a minimal visual barrier while maintaining safety standards. In this way, although the foyer, bar and restaurant areas are relatively small they do not have heavy boundaries and a feeling of spaciousness is created belying their actual dimensions. The upper foyer has two bars and is used as an exhibition area for local arts and crafts.

At either end of the upper foyer are the entrances to the auditorium. Adjacent to one of these is an external access for the disabled and wheelchairs, so avoiding the stair from lower to upper foyer. This was achieved by adapting the natural slope of the site.

The auditorium entrances are so placed that one enters at stage level either side of the stage. During performances this affords the players the option of using these entrances

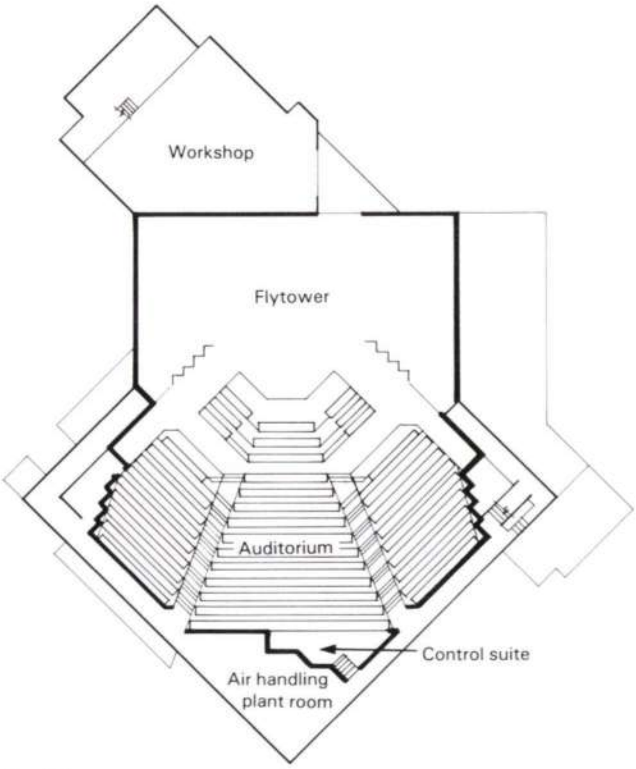


Fig. 13
Upper auditorium

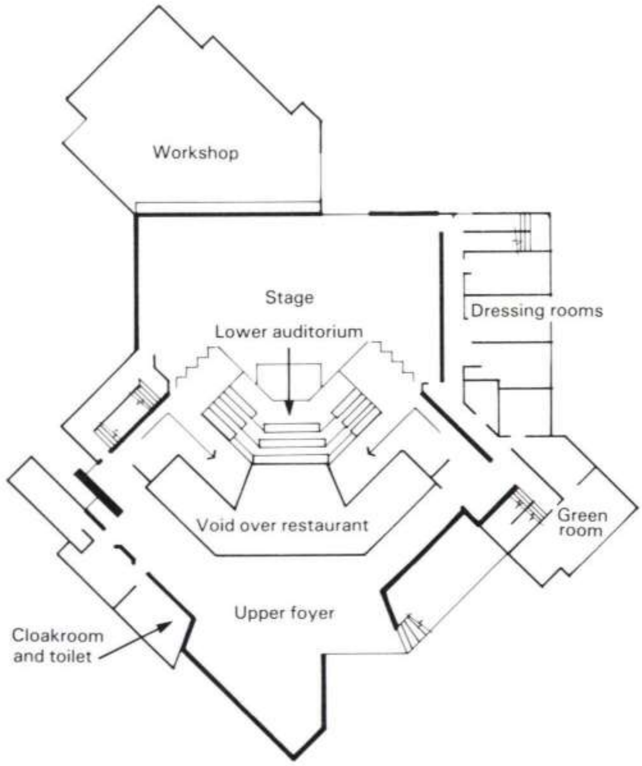


Fig. 15
Upper foyer level

Fig. 14
Glass façade at main entrance (Photo: Harry Sowden)

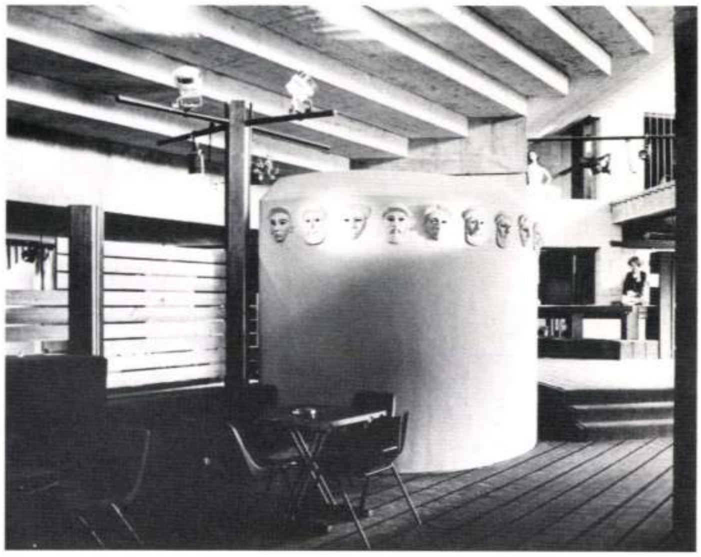


Fig. 16
Looking from lower foyer to box office (Photo: Harry Sowden)

in addition to conventional behind proscenium access.

The 450-seat auditorium has been designed to adapt to many different functions. At approximately two thirds from the front of the seating to the rear there is a low concrete blockwork wall separating the seats in front from those behind. For a small audience, therefore, if everyone was seated in front of the wall, the theatre would appear full thus preserving the proper atmosphere. It is possible to remove the front rows of seats which are set in a lowered area of the auditorium floor to create a pit for an orchestra for a touring ballet or opera. It is also possible to remove additional front seats, place them on the stage and create a theatre in the round.

In plan the auditorium is in the shape of a fan radiating from the stage. This form is echoed by the roof structure which has exposed hollow section steel trusses supported at the centre of the proscenium and diverging to the rear of the auditorium. In addition to supporting the roof of timber purlins and woodwool slabs, the trusses support two lighting bridges, again exposed, running the width of the auditorium. To the rear of the seating is a control room with a computerized console lighting palette, tape decks and follow spot. Behind the auditorium and above the upper foyer is the plant room housing the heating and ventilation plant.

There is a strong accent on flexibility of form which also applies to the stage that can extend by demountable panels into the auditorium. The head of the proscenium is defined by a frame clad in adjustable ply wood panels so that the entire frame may be set at any level above the stage and individual panels may be set up or down on the frame to create practically every conceivable profile. The sides of the proscenium are also adjustable, being formed by tubular, aluminium-framed periactoid towers triangular in plan with adjustable plywood panels on one face. The panels match the side walls of the auditorium and, by adding or subtracting towers, the proscenium width alters.

The stage is surmounted by a structural steel framed flytower clad in fairfaced concrete blockwork. A lighting bridge runs around the inside perimeter of the flytower at mid-height with a telestage grid floor giving access to the fly pulleys just below roof level.

To the rear of the flytower is the scenery workshop, well-equipped to manufacture scenery and props. Like the flytower this building is steel-framed with concrete blockwork walling. Both the workshop and the stage have a large external door. The two doors are adjacent and by opening each by 45° they become the side of a covered link between workshop and stage.

Stage left of the stage area are the theatre administrative offices with dressing rooms above. This is perhaps the only conventional part of the project, being a rectangular block with reinforced concrete slabs supported by

Fig. 17
Auditorium (Photo: Spanphoto, Dundee)

Fig. 18
Auditorium from stage (Photo: Spanphoto, Dundee)

load-bearing concrete blockwork. There are eight dressing rooms with showers en suite and a green room. 27 players can be accommodated.

The space heating and ventilation systems comprise two natural gas LTHW boilers rated at 367 kW each serving the following areas: one air handling unit for the auditorium and one air handling unit for the bars, restaurant and associated areas. The ancillary areas and dressing rooms are generally heated by fan convectors with local extract ventilation, with a small number of radiators in corridors. The workshop is heated with three unit heaters at high level. The domestic hot water is from two indirect storage cylinders heated from the main boiler plant.

The controls allow the auditorium ventilation system to be used independently from the rest of the building. Exposed oval ductwork with both supply and extract at high level blend into the auditorium with linear supply grilles running in parallel with the lighting walkways. The stage area has fan convectors round the perimeter hidden from the audience for use when rehearsals take place, thereby lowering the building running costs.

Low voltage electrical switchgear is housed in the lowest level of the building and serves the complete building by means of a network of armoured sub-main cables and trunking. The cable trunking installation generally comprises two-compartment trunking for mains and stage lighting circuits with single-compartment trunking, separated by 300mm enabling the accommodation of microphone cabling.

While the wardrobe and under-stage area is served by conventional fluorescent lighting, the lighting within the dressing room block is mostly tungsten, with bare bulbs round each mirror in true theatrical tradition. Lighting track is also used within the dressing room and administration block. The main auditorium is lit by means of tungsten spotlights mounted on the lighting bridges with two pygmy lamps incorporated in each riser of the stairs and connected to two circuits – one normal and one maintained essential circuit wired through the central battery unit, which also feeds the escape lighting in the auditorium. Emergency lighting in the auditorium is from a central battery unit and self-contained units are installed elsewhere. The workshop to the rear of the theatre utilizes high bay tungsten lighting in order that all stage sets and backdrops may be constructed with no problems from colour rendering of the artificial lighting.

The restaurant and two-level foyer make extensive use of multi-circuit lighting track such that by switching and dimming, the total character of the foyers may be easily altered to suit the varying requirements of the building. A complete system of fully dimmable stage lighting is incorporated, utilizing multi-gang outlet boxes mounted at stage level, gallery level and lighting bridge level. Stage working lights, blues lights, and rehearsal lights are also provided in the stage areas, galleries and fly tower. The total lighting and sound system within the auditorium and stage area is controlled from a computerized lighting pallette and sound rack within the control room at the rear of the theatre and provision has been made for the connection of outside broadcast equipment.

Conventional fire alarm equipment is installed with manual breakglass units and bells and automatic detectors are provided in the wardrobe and plant areas.

Simple finishes

Finishes internally and externally are simple. Walls are of oatmeal coloured concrete blocks. The auditorium slab soffit, the upper foyer slab soffit, dressing room and administrative ceilings, and entrance stair are all as struck concrete. The foyer and restaurant floors are carpeted as is the auditorium. The auditorium has black walls and roof soffit. Seats are black with thin red stripes. The only striking contrast in the auditorium is the provision of blockwork walls at the sides and rear of the seating and between the front and rear areas. These walls are lit by neon lights at their base. Throughout the building, the plan shape of the auditorium has been adopted as a motif and appears as the shape of external windows and numerous bits of internal decor.

The architect has endeavoured to produce a building which is not merely capable of functioning as a theatre but which has an interest value of its own and a place where people would like to be.

Credits

Pitlochry Festival Theatre

Client:
Pitlochry Festival Society
Architect:
Law & Dunbar-Nasmith
Quantity surveyor:
James D Gibson & Simpson
Theatre consultant
(including services and acoustics):
John Wyckham Associates
Main contractor:
J Fraser Construction

Dundee Repertory Theatre

Client:
Dundee Repertory Theatre
Architect:
Nicoll Russell, Architects Studio
Quantity surveyor:
D I Burchell & Partners
Theatre consultant:
Andre Tammes
Acoustician:
Sandy Brown Associates
Main contractor:
Burness & Son

Prestress design for continuous concrete members

Angus Low

Introduction

Despite extensive use of prestress in continuous members during the last 30 years, the design relationships for this form of construction are not widely understood. In a recent paper[1] it was shown that there are four cases which can govern the design, and an expression for the minimum prestress was derived for each. This paper extends the theory of the four design cases and reconsiders the basic criteria. The new-found understanding offers an opportunity to develop rational and practical design procedures.

History

Prestressed concrete is the brainchild of Eugene Freyssinet. His earliest studies date back to 1903[2] but the widespread application of his ideas was not possible until high strength steel wires were available after the Second World War. Freyssinet was clear that prestressed concrete was more than an enhanced form of reinforced concrete. He had a practical understanding of the separate needs of durability and safety and he put great emphasis on the ability of prestressed concrete beams to carry a full range of working loads completely uncracked but, if an overload occurred, there would be a rapid and noticeable development of cracks and an abrupt increase in deflection while the member still had some reserve of strength. Early practitioners limited the working loads to the range which would cause no calculated tension in the concrete. The design requirements to achieve this condition in a simple beam were best demonstrated by Gustave Magnel in his Magnel diagram (Fig. 1). The diagram shows the prestressing force plotted as its inverse 1/P against the position of prestress within the section. He showed that the four limiting conditions of zero stress and maximum permissible stress in the top and bottom fibres are represented by straight lines on the diagram and enclose a quadrilateral region of acceptable designs. For a given force P the position of the cable must lie within the 'cable zone' ZZ. The minimum prestress design is represented by the Point X. Two design cases exist depending on whether X is within the cover limits of the section or not. When X is within the cover limits, the design of the concrete section is governed only by the range of live load moments applied and is independent of the dead loads. This notion was given undue prominence and prestressed concrete became known as the material in which it was only necessary to design for live load; the dead load carried itself. It followed from this understanding that joining multiple beams into a continuous structure offered no benefit. The savings in live load span moments due to continuity were offset by the increase in moment range due to moment reversals when adjacent spans were loaded. Unlike reinforced concrete, prestressed concrete has been used mainly for simply-supported spans.

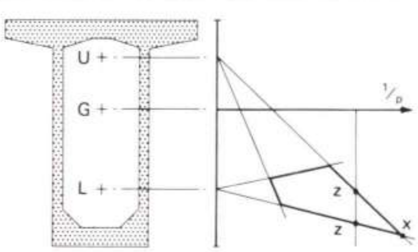


Fig 1
Magnel diagram

Fig. 2
Arup-designed walkway at the Festival of Britain South Bank Exhibition in 1951 (Photo: Planet News)

Perhaps the most obvious application to benefit from prestressing is bridge construction. Structural continuity offers many advantages to bridges which are not related to the characteristics of the construction material. There are savings in joints and maintenance, improvements in ride for the road user, simplifications in the transfer of horizontal loads and benefits in construction. Prestressing has been used for many years in continuous multi-span bridges and viaducts. One of the earliest continuous prestressed structures was an Arup-designed walkway at the Festival of Britain South Bank Exhibition in 1951 (Fig. 2).

The application of prestress to a continuous beam sets up an additional set of reactant moments in the beam called parasitic moments. These parasitics upset the simplicity of Magnel's diagram. Designers managed to retain his theory by considering only 'concordant' cable profiles which produced zero parasitic moments. These profiles were generated directly from the moment diagrams of applied loadings and their application was widened by the use of 'linear transformations'. However, these procedures were iterative and indirect. Except in the case of moment range governed designs (design case 1), there was no qualitative way of understanding how changes to the loading or the shape of the concrete sections would affect the required prestressing force. Later it was realized how useful the parasitic moment could be to the designer. In a continuous member it could be used to redistribute moment between the support and the span to maximize the effectiveness of the prestress (design case 2). The optimum redistribution would imply a certain parasitic moment to be achieved during detailed design of the profile. Unfortunately in some situations, typically in a ribbed slab with slender ribs, the implied parasitic could not be achieved even with the cable tight to its limits along its full length. It is the design relationship for this last case that has been published recently (design cases 3 and 4) and there is now a complete theory which relates the required prestress to the loading and the shape of the concrete member.

Theory

The recent paper derives expressions for the minimum prestress of an internal span with a varying section and variations of cable force are also considered. Both T and trough sections are discussed and the effect of permitted tensions is mentioned. With so many variables the simplicity of the design relationships is lost. By considering only uniform members with equal spans the four relationships can be re-expressed very succinctly. The basic components of the concrete section are shown in Fig. 3. The kerns are the upper and lower limits of the core region which is the generalized 'middle third' of the section. The total load and live load moment envelopes are shown in Figs.

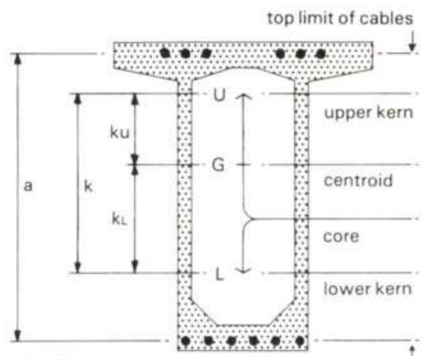


Fig. 3
Section geometry

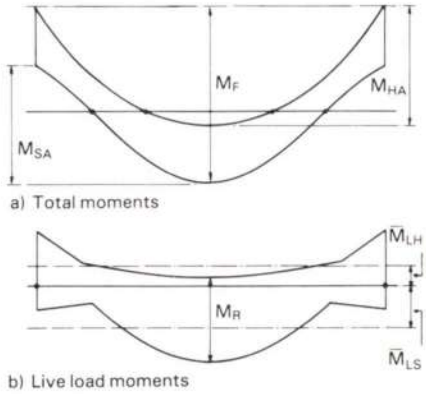


Fig. 4
Moment envelopes

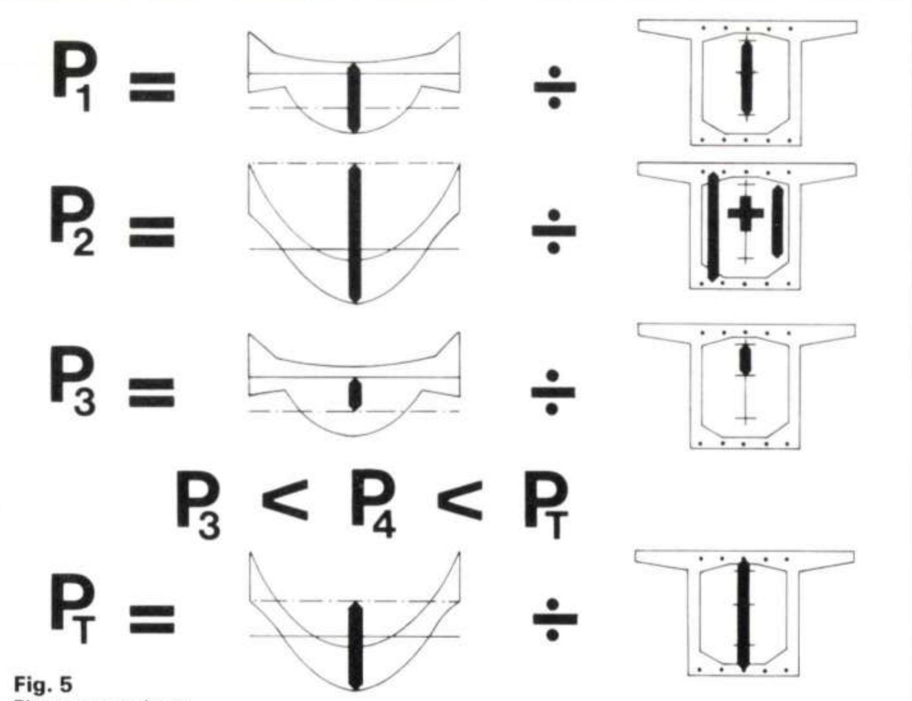


Fig. 5
Picture equations

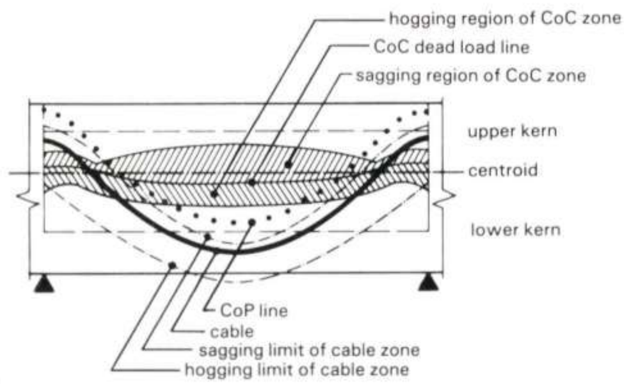
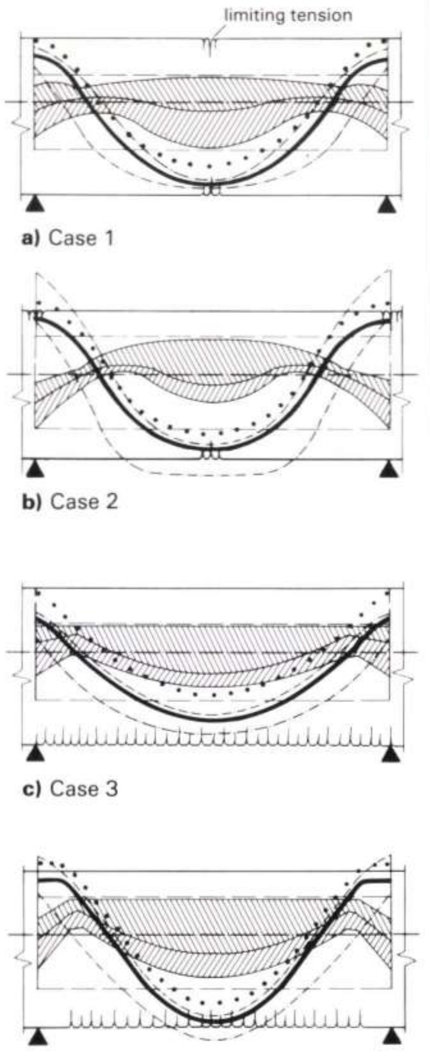


Fig. 6
The hidden anatomy



a) Case 1

b) Case 2

c) Case 3

d) Case 4

Fig. 7
The four design cases

4(a) and 4(b). $\bar{M}_{LS}$ and $\bar{M}_{LM}$ are the mean values of the sagging and hogging live load moment envelopes in the span. The required prestress for the four cases can be expressed as the ratios of dimensions from the moment and section diagrams and the picture equations demonstrating this are shown in Fig. 5. The prestress P required in design is the highest of the values P_1 to P_4. The reasoning below derives these geometric relationships directly.

The hidden anatomy

Fig. 6 shows a longitudinal section of a typical span of a continuous prestressed member with considerable vertical exaggeration for clarity. Moments at any section in the span are resisted by the couple acting between the centre of prestress CoP and the centre of concrete stress CoC. With no axial load in the member the total concrete force is equal to the prestressing force P. The profile of the CoP is fixed by the designer and the profile of the CoC dances up and down in response to variations in the applied load. Its dance sweeps out a zone shown shaded in the diagram. An eccentric CoC implies there is curvature in the member and in general a net rotation along the length of a span. In a continuous member with equal spans under equal loading there cannot be a net rotation in any span so the mean of the CoC profile must be at the level of the centroid. If all loads including gravity are removed then the CoP and CoC profiles must coincide. It follows that the mean of the CoP profile must also be at the centroid. It is the parasitic moment that shifts the CoP from the cable profile to the position where this is achieved. If the spans are identical the parasitic moment is the same at every support and so is constant along the span. The shift from the cable profile to the CoP profile is uniform without any slewing.

If the 'no tension' criterion is applied, the CoC must always be within the core for all service conditions. Notice that at any section the depth of the unused area of the core between the CoC zone and the kern is the same as the gap between the cable and the cable zone limit.

Design relationships

Case 1

The amplitude* of the dance of the CoC is M_R/P where M_R is the live load range of moment (see Fig. 7a). The prestress P_1 necessary to enable this amplitude to fit into the core depth k is given by:

$$P_1 = M_R/k$$

Case 2

Variation in moment along the member is taken in both the CoP and CoC profiles. To achieve the maximum load-carrying capacity of the beam, the lever arm of the resisting couple is maximized both at the support and in the span. The full amplitude of applied moments along the span M_F is resisted by the full available amplitude of the CoP profile, a, together with the full amplitude of the CoC profile, k. The limiting prestress force P_2 to achieve this maximum load condition is given by:

$$P_2 = M_F/(a+k)$$

*'Amplitude' is used to denote full depth from peak to trough.

Case 3

In choosing the profile of the CoP the designer is also fixing the ambient (Dead Load) profile of the CoC. Because there is no net rotation in a typical span the mean of this profile must be at the level of the centroid. In sections where dimension k_u is small it is difficult to meet this requirement while also keeping the CoC zone below the upper kern. The limiting condition is shown in Fig. 7c. The area of the sagging CoC zone is $\bar{M}_{LS}L/P_3$ where $\bar{M}_{LS}$ is the mean value of the live load sagging moment envelope in the span. This is equal to the area of core above the centroid k_uL. Hence the limiting prestressing force P_3 is given by:

$$P_3 = \bar{M}_{LS}/k_u$$

Case 4

In Fig. 7c the CoC profile for maximum sagging moments has no amplitude. The variation of the sagging moment envelope along the span M_{SA} is taken on the amplitude of the CoP alone. This is only possible if M_{SA} is smaller than P_3a. For larger M_{SA} it is necessary to take some of the amplitude on the CoC (see Fig. 7d). This requires additional prestress:

$$P_4 > P_3$$

This increased prestress is not enough to dispense with the need for some amplitude from the CoC sagging limit.

Hence: $P_T > P_4 > P_3$

where $P_T = M_{SA}/a$

The evaluation of P_4 within this interval is given in the earlier paper.

Fig. 8
Cable zone

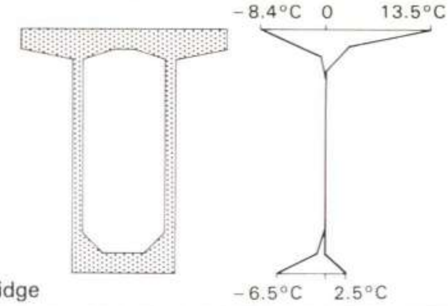


Fig. 9
Temperature gradients in a bridge

Detailed design

The earlier paper is limited to preliminary design. Because the required prestress is derived directly during preliminary design the detailed design of the cable profile does not need the usual protracted iterations. It is clear from the above understanding of prestressing in a continuous member that the effect of a cable profile will depend on its shape only and is not dependent on its level. Whatever the cable level the CoP will take its shape and shift it onto the centroid as its mean line. It follows that the shape of the profile can be derived at any level in the section and then be shifted up or down to fit within the section. For convenience, assume the cable is first drawn at the level which gives zero parasitic moment. In this case the cable zones can be drawn quite simply. In the trivial case of no applied moments it is the kerns that are the limits of the cable zone. The upper kern is the limit outside which the section would suffer sagging tensions and the lower kern is the limit for hogging tensions. The effect of applied moments is to shift these limits so that the upper limit becomes an image of the sagging moment envelope divided by P hanging on the upper kern. Similarly the lower limit is the hogging moment envelope divided by P hanging on the lower kern (see Fig. 8).

For cases 3 and 4 the cable follows tight along the edges of the zone or section and so is defined uniquely. For cases 1 and 2 it is necessary to remember that a zero parasitic moment has been assumed. The profile chosen must have its mean level on the centroid. Once drawn the cable is then shifted to its real position. The effects of friction and other losses must be introduced into the analysis and a detailed check is required. This check is likely to require some fine tuning of the profile.

Design criteria

To develop rational design procedures it is necessary to understand both the design relationships and the design criteria. A rational approach to design criteria is now usually called 'limit state design' and it distinguishes between the requirements at 'ultimate limit state' (ULS) and 'serviceability limit state' (SLS). If a prestressed section were loaded to failure its ultimate moment would be very close to that achieved by the same section which had the same cables grouted into its ducts but not prestressed. Prestressing has nothing to do with the ULS. The purpose of prestress is to enhance the behaviour of the beam at SLS. In particular it is needed to restrict the cracking of the concrete to an acceptable level. In bridges which are subject to fatigue loading the slightest cracking around the cables can be significant.

Most structural research and theory during the last 30 years has been concerned with ULS behaviour and safety. The great advances in this field are due in part to the prudent preference of engineers for materials which sustain their ultimate strength over a wide range of strain. Strength is independent of strain and ULS analysis can ignore the unquantifiable complexities of strain that exist in real materials. This is not true at SLS and prestress design is all about quantifying the unquantifiable. Traditionally in prestress design the tensile strength of concrete has not been quantified either and design has proceeded on the optimistic hope, backed up by empirical observations, that the two unknowns will cancel.

Before leaving the ULS there is one current practice which needs to be corrected. Although the ULS does not govern the prestress design the prestress does participate at ULS. In the ULS check the actions are factored and marshalled into opposing ranks of 'load effects' and 'resisting effects'. The confusion that commonly surrounds prestress is demonstrated by the fact that prestress is usually required to fight on both sides. The free component is classified as a resisting effect and factored down. The parasitic component is added to the loads with a nominal factor of unity. The equivalent load method, which of all analysis methods models the prestress the most directly, makes no distinction between the free and parasitic components and so cannot be used without elaborate manipulations. As parasitic moment is a strain compatibility effect its participation at ULS is questionable and in the cause of simplicity all prestress should should be included with the resisting effects.

A number of strain effects which exist in prestressed concrete members are listed below:

(1) Differential temperature

Radiation from the sun during the day and the ground at night sets up temperature gradients in a concrete member. These gradients have been measured and most bridge design codes require their assessment. Fig. 9 shows a typical temperature distribution range from *BS 5400*[3]. These can result in a stress of about 3 MPa.

(2) Creep redistribution

When a member is cast in sections its built-up pattern of bending moments can differ significantly from that for the member cast as a complete structure. After completion the moment pattern will creep from the 'built-up' condition towards the 'complete structure' condition. The extent of this creep is sensitive to the speed of construction and details of the concrete mix.

(3) Differential shrinkage due to stage construction

When a concrete cross-section is cast in stages the shrinkage that has already occurred before subsequent stages are cast will result in differential strains in a member. This can give stresses of about l MPa.

(4) Differential shrinkage due to variations in thickness

Thin sections shrink and creep more and respond faster than thick sections. It has been shown by Bryant & Fenwick that in the bridge section shown in Fig. 10 which has little variation in thickness this effect results in stresses of 0.5 MPa.

(5) Surface shrinkage

Surfaces of members dry out more readily and hence shrink more than the interior.

(6) Thermal peak stressing

This is a technique that can be applied during construction to alleviate cracking due to some of the above effects. The heat of hydration in the concrete causes a significant temperature rise which reduces towards the surface of the member. The peak temperature is reached after about two days and at this stage the warm fresh concrete sustains very large creep strains under load. If a light level of prestress is applied at this age the stress will pass preferentially through the interior of the member where the thermal strains are greatest. This hot stressed core is also the most sensitive to creep so there will be a large creep strain differential between core and surface. After cooling and final stressing this results in a member whose surfaces are more highly prestressed that its interior.

Under current practice, estimates are often made for items (1), (2) and (3) above but (4), (5) and (6) are not usually considered. The distinction is arbitrary and so the stresses that appear in calculations are to a similar degree arbitrary. In any design criterion

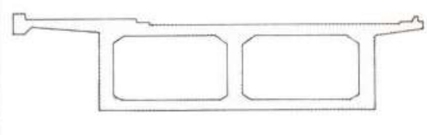
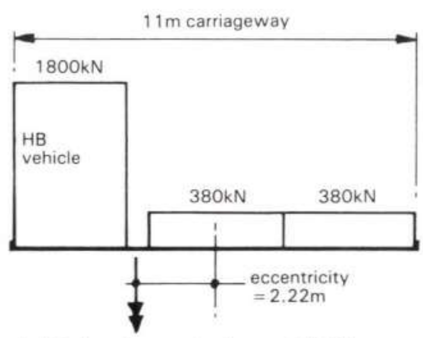
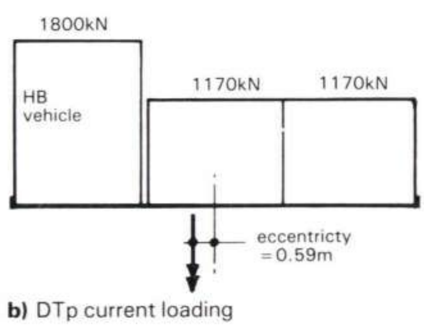


Fig. 10
Bridge section
used for differential shrinkage study



a) DTp loading up to August 1982



b) DTp current loading

Fig. 11
Live load eccentricities under maximum highway loading in a 40m span

which refers to the stresses in the section it is important to define how the stresses are calculated. The seemingly constant 'no tension' criterion which has been used since the time of Freyssinet has become increasingly conservative as more component stresses have been included. The current move towards partial prestressing in which tensions less than the tensile strength of concrete are allowed is really a rationalized return to the original 'no tension' criterion.

At a time when design criteria are changing it is important to understand the real consequences of change because, as has been shown, criteria cannot be read at face value. This is why a clear expression of design relationships is so important. Currently two changes in bridge design are both favouring a swing from box sections to double T decks. Firstly partial prestressing, introduced in the UK in *BS 5400 Part 4 1978*, increases the effective core depth k_e.

$$k_e = k\,(1 + A\,\sigma_T/P)$$

where A is the section area and σ_T is the permitted tension stress in the concrete. The advantage of partial prestressing is proportionately less for case 2 so this design case is more likely to govern. Box sections offer a large core depth k in exchange for construction complexity. The large core depth is not so important for case 2 and so the simpler double T is more likely to be chosen. The second reason for choosing a box is because of its torsional efficiency. A recent change in Department of Transport highway loading has greatly reduced the maximum torsions due to live loads (Fig. 11) so again the advantage of the box is lost.

An alternative to the usual 'permitted tension' version of partial prestressing is the 'permitted depth of tension' method. This has advantages because many of the strain effects are most dominant near the surface and also the design relationships are slightly simpler because the expression for k_e is independent of P. It also recognizes the fact that cracks are acceptable provided they do not extend to a depth sufficient to threaten the life of the cables.

Design procedures

The dependence of prestress design on the complex strain behaviour within sections has far-reaching implications for design procedures. Either such complex behaviour is covered with conservative values for the tensile strengths of concrete or savings can be made by estimating each effect. Many of the effects depend on details of the construction procedure and could only be included if detailed design and detailed construction planning were carried out simultaneously.

Conclusion

The theory of prestressed concrete design (as opposed to behaviour) is still in its infancy. There now exists sufficient understanding of design relationships and the behaviour of members to develop practical design procedures which come nearer to realizing the full potential of prestressed concrete.

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Arup Acoustics

Job no. AA292
Henry Wood Hall, Southwark

On p.23 of The Arup Journal, December 1982, a photograph of St. Johns, Smith Square, was wrongly captioned as the Henry Wood Hall, Southwark. Two photographs of the real Henry Wood Hall, are included here.

Fig.1
Acoustic test of Holy Trinity Church, Southwark, in December 1972, before conversion into Henry Wood Hall.

Fig.2
Henry Wood Hall in February 1976, after conversion into a rehearsal hall. (Photos: Arup Associates)

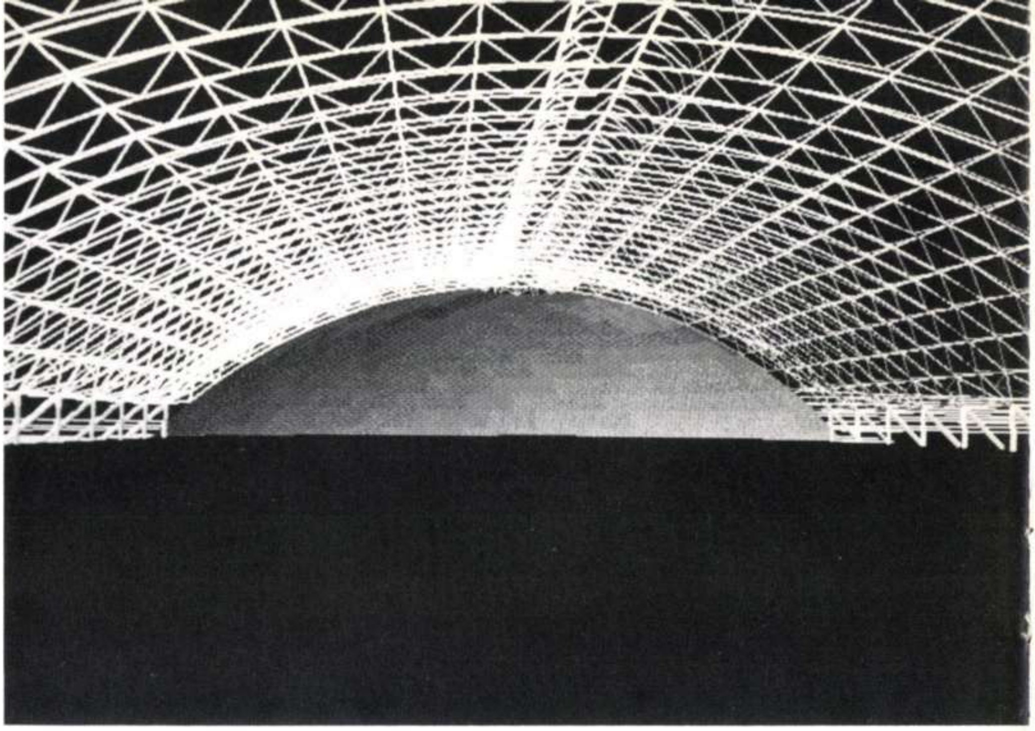
Merseyside Garden Festival 1984: Festival Building

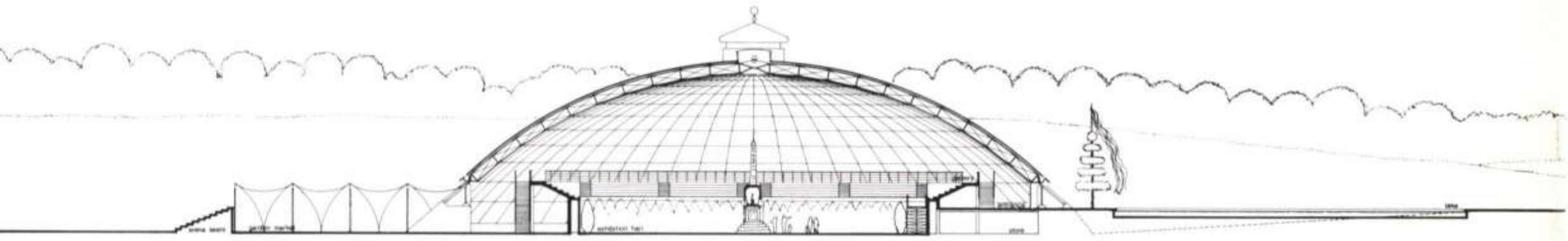
Arup Associates Group 4

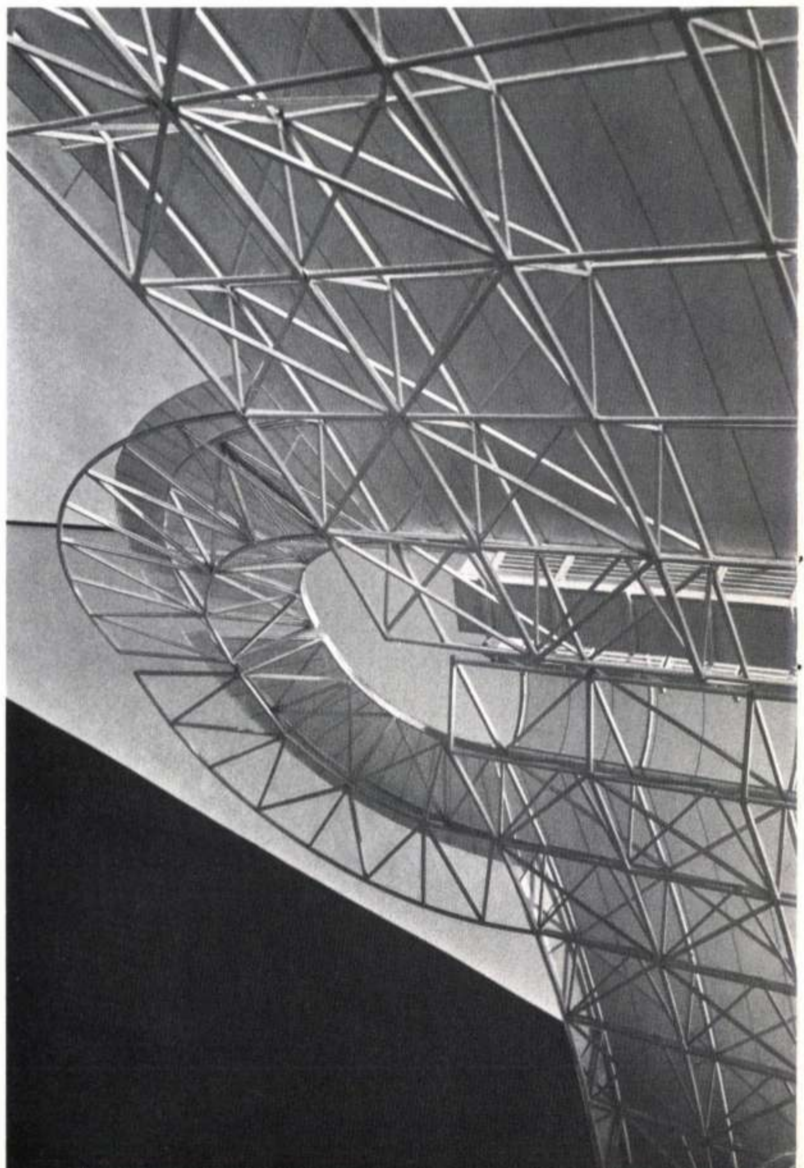
Client: Merseyside Development Corporation

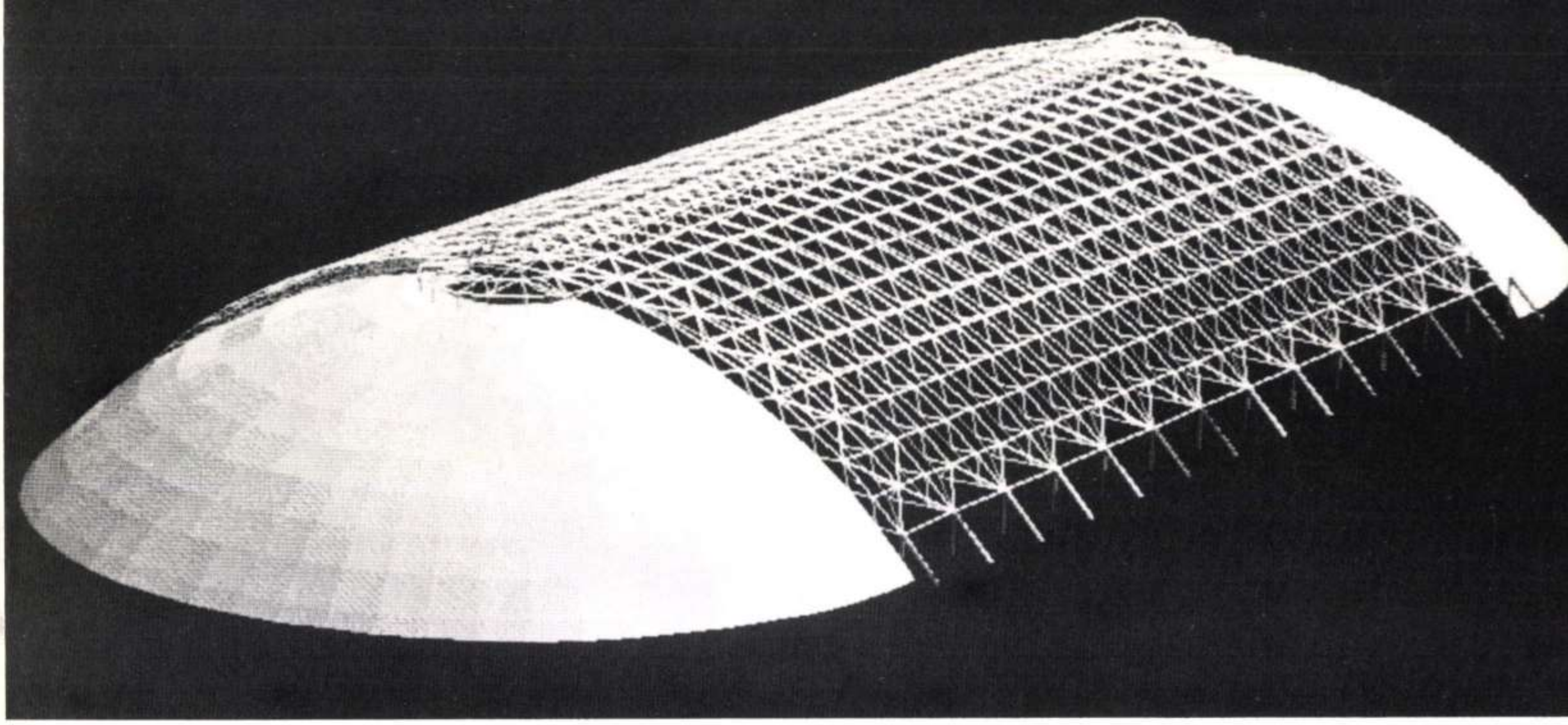
Competition winner: July 1982
Start on site: January 1983
Completion: March 1984

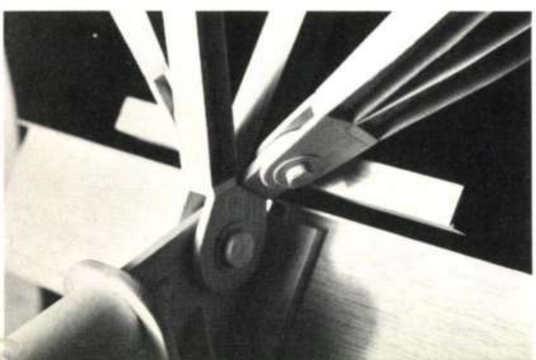
The building is designed to become
A sports and leisure centre for
Liverpool City Council
after the Festival.

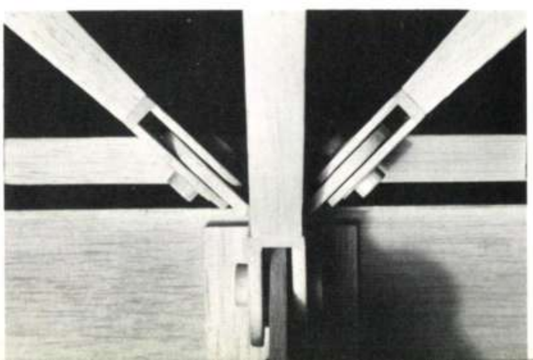


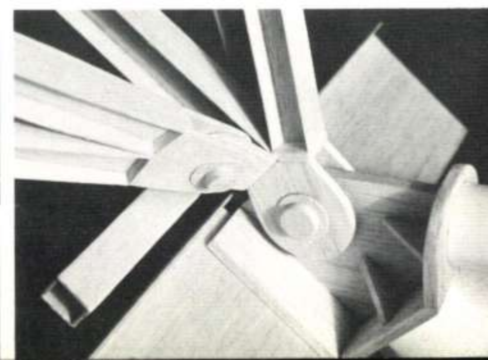


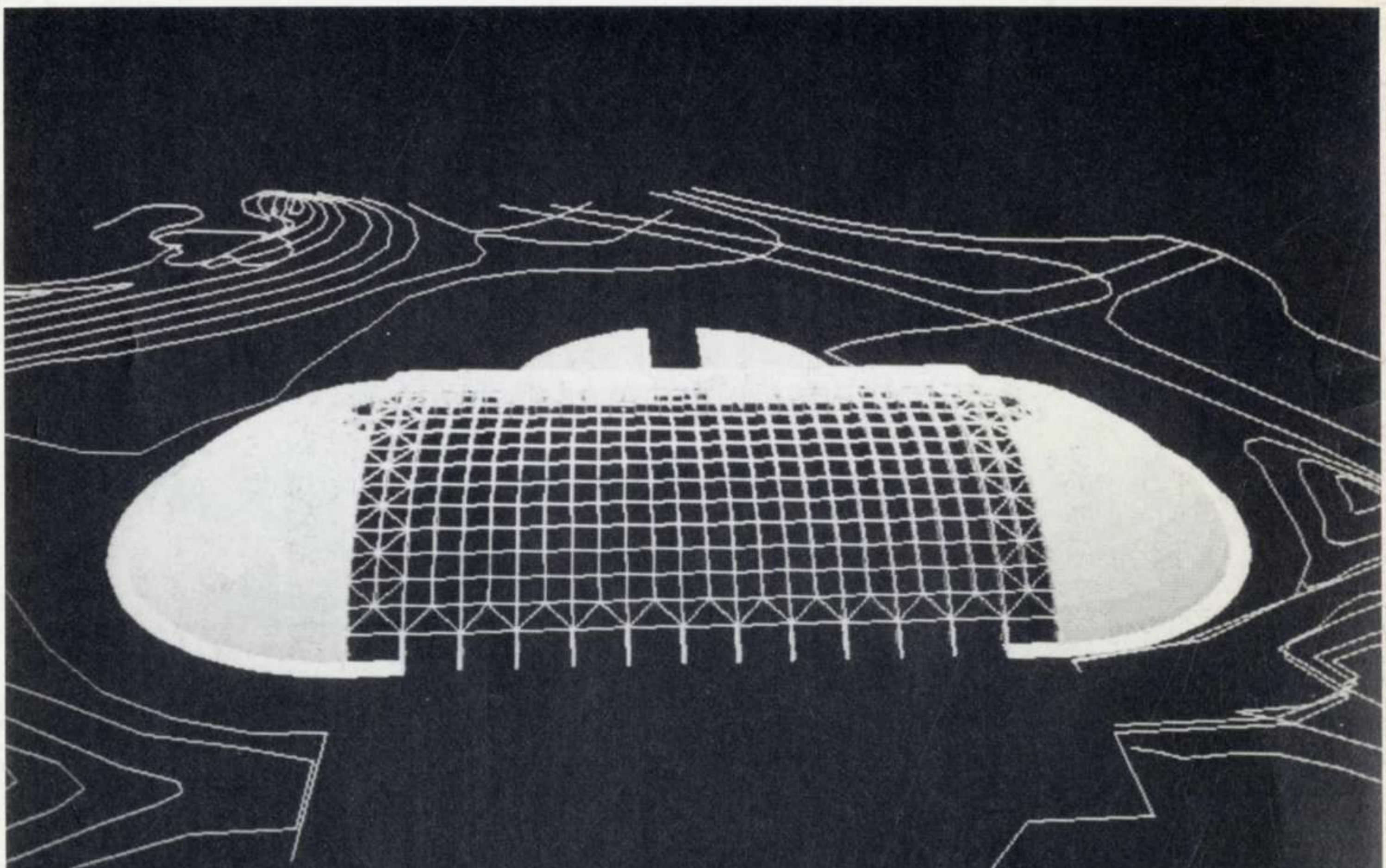












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