Section II - Case Studies

EDINBURGH COLLEGE OF ART
HERIOT-WATT UNIVERSITY
DEPARTMENT OF TOWN PLANNING
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Introduction

1.0 Volume II of the thesis comprises selected case studies illustrating both key aspects of structure and architecture in relating structural development to form and space.

These studies are described in the following context:

i) architectural considerations

ii) structural considerations

iii) utilitarian considerations

iv) construction considerations

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2.0 Buildings were compared on the basis of their relation to arch or cable-stayed structures, although functions and scale varied from one to another. There is perhaps a correlation between achieving increasing lighter and visually more slender structures with the application of tension and compression systems in preference to those which operate in bending. Although this may be a valid theoretical ideal, there are very few pure compression or tension structures in practice nor is there a total elimination of bending. The case studies illustrate the factors and reasons which lead to the antithesis of this design ideal before arriving at conclusions detailed in Section III of the thesis.

3.0 The first group of case studies comprise the application of the structural arch system and the second group comprises the application of cable-stayed systems. The third group of case studies are combinations of arch and cable-stayed systems, and

the fourth group comprises miscellaneous systems. The Case Studies are summarised in respect of the four main considerations aforementioned, in Volume I Section II of the thesis.

SECTION II - CASE STUDIES MASTER LIST

Group 1 - ARCH and PORTAL SYSTEMS

1 Frankfurt Athletic Stadium
2 Liverpool Festival Hall
3 Embankment Place Development
4 Lee House Development
5 Broadgate Phase II
6 Battlebridge Basin Proposal
7 Imperial War Museum, Southwark
8 Lloyd's Roof Atrium
9 Sainsbury Camden
10 Debden Printing Works
11 Patera Nursery Units
12 IBM Sports Hall
13 IBM Travelling Exhibition

Group 2 - CABLE-STAYED SYSTEMS

14 Shotts Factory
15 Renault Centre Swindon
16 Fleetguard Factory, Quimper
17 Factory at Nantes and Epone
18 Oxford Ice Rink
19 Ladkarns Building
20 Patscentre
21 Sydney Football Stadium
22 Crystal Palace Extension
23 INMOS Factory
24 Homebase Brentford

Group 3 - ARCH and CABLE-STAYED SYSTEMS combined with OTHER SYSTEMS

25 Bridge of the Future
26 Austerlitz bridge - competition entries by Rogers, Foster and Calatrava
27 Stansted Airport
28 Charing Cross Proposals by Foster Associates
29 Glasgow Eurodrome competition entries - Whitby, Marks & Barfield, McCarthy and Potter
30 Selected works of Santiago Calatrava
31 Kansai Airport
32 Hambourg Ferry Terminal
33 Ravenna Stadium

Group 4 - MISCELLANEOUS SYSTEMS

34 Fujisawa Stadium
35 Tokyo Metropolitan Centre
36 Bari Football Stadium
37 Stuttgart Art Gallery
38 Berlin Bismark Strasse Industrial Development
39 Blackheath Meeting House
40 Hongkong and Shanghai Bank
41 Sho-Hondo Temple
42 Les Tours de la Liberte
43 National Indoor Stadium, Singapore
44 Barcelona Stadium

GROUP 1

CASE STUDIES 1 AND 2: THE FRANKFURT ATHLETIC STADIUM AND THE LIVERPOOL FESTIVAL HALL

Utilitarian Demands

The Frankfurt Athletic Stadium called for an enclosure of a 200 m track, 2 sprint tracks, space for minor events and seating for 3000, changing rooms and administration seats. The client is presently considering the detailed architectural and engineering design submissions.

Physical Dimensions

140 m x 70 m.

69 m span between concrete abutments. The wall rises 9 m at its crown to give a maximum floor to roof clearance about 15 m.

The Liverpool Festival Hall was a simple brief for three exhibition halls each of which would be used separately or together in any combination. Its alternate function as a sport complex required it to accommodate three sports halls, six squash courts, a leisure pool, a practice hall, a projectile hall and seating for 4000.

Physical Dimensions

126 m x 60 m.

Half spherical dome 30 m radius.

Implications on Design Options

Instead of designing a specific building to satisfy the sports complex, omitting parts for the festival phase, Arup Associates decided on a "loose fit" building for the Liverpool Hall, which could take a great deal of variation within the brief (Figure 14). The

the Frankfurt Stadium) required space free from obstruction to sight-lines and seating structures.

In both buildings, structural supports at ground level were thus to be confined to the peripheral zones of the floor plan (Figure 1B, 4).

Roof Loads

In addition to normal design loading, the Waldstadion roof structure was required to support point loads from a suspended press box (Pg U9, OAP Report "ANALYSIS AND DESIGN OF A PREFABRICATED THREE-WAY TRUSSED GRID VAULT", Zunz, Hough, Banfi).

The Liverpool Festival Hall was designed with the intention of dividing the interior space at the main divisions in the structure. Canvas and glass "screens" were to be suspended from the roof structure.

These requirements implied that the roof structure at the points of attachment of the divisory elements had to be specially braced to carry the localised loads.

Site Context and Character

Both the Frankfurt Stadium (Waldstadion or Stadium in the woods) and the Liverpool Festival Hall were sited in natural surroundings. The Festival Hall Building was to reflect the mood of festivity and was conceived as a shell-like a theatre, within which the widest possible range of festive events could be mounted. The design of the garden festival building was to be perfectly balanced with the scale of its surroundings.

This implied a need to avoid the introduction of structures which created a visually bulky image. The emphasis on the visual links between exterior and interior space and the provision of a generous flux of daylight through the roof depended on achieving slenderness in the structural members.

Choice of Form (or shape) of Building

In the case of Liverpool Festival Hall, the use of simple geometric forms in harmony with the surrounding landscape was thought to provide the visual clarity required to unite the separate small-scale elements within the festival park. As a backdrop to the study of plants, it had many metaphors ranging from a land form to the Crystal Palace.

The form was preconceived as the union of a barrel vault and two hemispherical domes (Figure 2A, 2B, 3). Although the resultant form suggested visual simplicity, the union of the two geometric forms did not simplify the structural solution. The path of the loads transmitted along the suggested geometry, changed at vault and dome junctions and this required structural attention (Figure 4).

Structural elements were arranged out of the following considerations:

a) A compatible geometry was required to unite the domes and the vault.

b) The integrity of the interior space was to be preserved without any interior structural projections which would appear to visually divide it. The function of the dome and vault systems was thus expressed externally (Figure 4).

The vault system comprised planar trusses spanning the shortest distance whilst the dome ribs comprised single I - section steel joists cast to the correct curvature.

For this reason, the dome ribs were continuous with the lower chords of the vault trusses and tied longitudinally by purlins at that level.

c) A convenient and regular structural grid was required for the attachment of cladding - transparent polycarbonate sheeting for the vault and aluminium panels for the end domes.

Relatively slender purlins (longitudinal members) were permitted with a 3.0 m bay of planar trusses.

d) Evenly distributed (building) dead and live loads were planned for the structure which had to absorb differential settlements from a filled site.

By contrast, the choice of the Waldstadion form was made out of the following structural considerations in relation to its extremely tight site constraints:

The need to preserve a flanking row of trees on one side of the site and the need to provide a concourse of adequate width (outside the structure) further constrained the 140 x 70 m ground floor plan (Figure 5A).

This offered little opportunity for two-way structural spanning and so the range of one-way systems was studied (Figure 5B, 6, 7).

The relative COSTS of the alternative systems considered could be summarised as follows:

Type	Cost Rating
i) Circular steel plate monocoque with diaphragms separating two stressed surfaces.	Most Expensive
ii) Rectangular trussed portals.	
iii) Simply - supported trusses.	
iv) The elliptical arch with its efficient use of interior space but with every joint of different fabrication geometry.	
v) Cranked portals.	Lightest and least expensive
vi) The circular trussed arch	

The cost comparison revealed that the cross-sectional configuration closest to the <u>inverted funicular line</u> with the minimum bending that could develop in the structural members, would be the least expensive.

The circular trussed arches spanned 69 m between concrete abutments, and the vault rose 9 m at its crown giving a maximum floor to roof clearance of about 15 m.

The <u>bonus</u> of the circular shape was the constant joint geometry and the ease of prefabrication.

The structural depth (centre to centre of chords) would be limited to 1100 mm. This dimension allows for HVAC and electric ductwork clearances. the challenge to span the distance was not entirely done to a budgetary constraint but that the provision of a generous degree of daylight depended on achieving very light structural framing.

Choice of Framing Pattern

Although the simplest choice for a circular vault structure was a set of well-spaced, parallel main trusses spanning the shortest distance joined with longitudinal purlins, the Waldstadion designers sought a more elegant and visually interesting solution through its framing pattern.

New patterns for the structural grid were studied to counteract the problem of buckling with slender members, by introducing a high frequency of longitudinal bracing.

Coupled with the architect's preference for discrete panel-type cladding and roof lighting units the structural grid was manipulated from trusses in a short-span direction into a two-way diamond grid. This two-way grid had a 'spreading' tendency and frequent longitudinals to stop this resulted in a three-way grid. The main chord members were to carry mainly compressive forces (being part of an arch system) and were thus subject to buckling

Three strategies were possible in counteracting buckling, all these being related to the slenderness ratio (l/r).

a) Reducing the effective length of members.

b) Increasing the cross-sectional area of members.

c) Altering the cross-sectional shape of the members.

Strategy (b) was opposite to the idea of creating slender elements and not considered.

Strategy (c) was not practical in terms of joint fabrication and the convenience of using hollow-section steel members.

Strategy (a) was adopted: the decision to offset the longitudinals instead of passing them through the main chord reduced the effective length of all chords, and created large hexagonal cells which permitted a range of visually different and interesting panel types (Figure 8).

The hexagonals were 3 m across faces and 1.732 m on a side, these lengths giving the structural members visually attractive slenderness ratios which required only minimal permissible stress reductions (Figure 9, 10).

Fixing the main chord patterns by no means fixed its web geometry. 'Offset' webbing would be visible on plan view was considered but the 'in-line' double Warren webbing allowed ventilation ducts to pass through it along offset lines of the same grid.

Gable-End Articulation

In the Waldstadion, the architect was interested in articulating the gable ends of the vault, in a way which reflected the diamond shaped grid. One consideration was given to projecting the long sprint track

out one end and another given to reshaping the entry area.

In any case, the projection of gable-ends from the barrel vault frames were structurally, longitudinal cantilevers. Gable-post propping was also considered to achieve the architectural effect at the ends of the vault and structural guidelines given to "rationalise" the random zig-zag gable edges (Figure 11).

In the Liverpool Festival Hall, the straight-forward framing pattern of the barrel vault (with planar trusses spanning the shortest distance) was adopted.

The structure divided itself naturally into three parts: the two dome sections connected by the vault (Figure 4).

The structural problem posed by the combined forms was not as straight forward as the architectural idea of combining vault and dome

This was because the vault required only horizontal bracing in order to provide stability, the dome sections required horizontal and vertical support at their discontinuous vertical edges.

The initial studies of the dome discontinuity support showed that stability could not be achieved within the dome structure itself without massive and quite out-of-proportion members.

It was therefore decided to use the continuation of the ventilation structure (which was also a dominant architectural feature), to resolve this problem.

The principle here was to introduce a semi-circular (in plan) triangular (in section) braced girder which would cantilever from the last three vault frames and pick up the braced-in-plane ends of the dome ribs. consequently, these three end frames were treated as braced bays with in-plane and out-of-plane bracing providing horizontal and vertical reactions respectively to the out-of-balance forces arising from the dome discontinuity (Figure 12).

Surface Continuity

The plane of the dome ribs was made continuous with the lower chord of the vault because the underside of the vault was to be kept as a single unbroken line along its length and also because the underside of the connected domes were to be a continuation of the underside of the vault.

Support Spacing and Truss Spacing

The vault frames (planar trusses) were spaced at three metres with alternative frames being supported off the adjacent lower pins, with a bifurcated braced frame. This gave a buttress spacing of six metres which saved on the cost of otherwise additional support and opened up the lower sides of the vault, creating an effect of visual lightness (Figure 4, 13).

CONSTRUCTION CONSIDERATIONS

Fixed vs Pinned-connections

The arch system of the Waldstadion was designed with FIXED supports due to preference for a higher safety factor accorded by the design guidelines against buckling (Pg 7, OAP Design Report "ANALYSIS AND

DESIGN OF A PREFABRICATED THREE-WAY TRUSSED GRID VAULT", Zunz, Hough & Banfi).

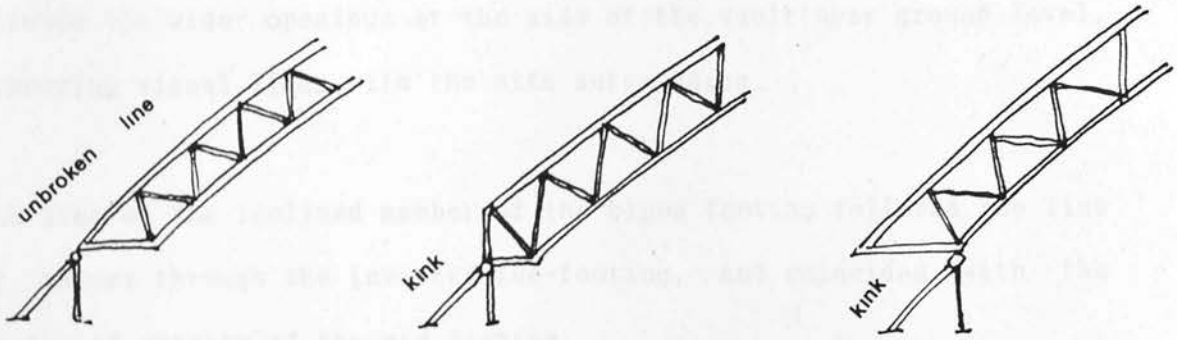
The advantages were greater stiffness of members with minimum deflection. the rotation of concrete abutments due to differential settlement was discounted with fairly dense sand and gravel ground conditions.

The choice of "fixed" springing points to the arch (rather than "pinned"), provided additional redundancy against overload in the transverse direction and was thus very safe from progressive collapse (Figure 14). The advantages were that higher peak moments resulted with asymmetrical loading and shear distortion (due to thermal movement) between the steel frame and the concrete abutment supporting its edges.

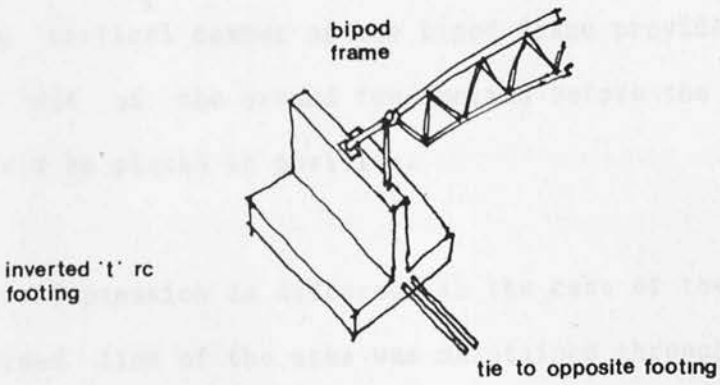
By contrast, the Liverpool Festival Hall vault frame adopted three-pinned arches which would be less sensitive to differential settlement of the foundations, laid on compacted fill. Pinned connections allowed both on an off-site fabrication and facilitated easy erection (Figure 15).

The disadvantages were a marginal (10%) increase in the maximum bending moment with the three pins on the axis of the top boom compared to the structurally ideal mid-point position. This increase did not cause significant changes in member forces to result in a change of section size.

The location of the pin joint relative to the top and bottom chords of the trussed arches was influenced by the architectural consideration of maintaining the curved line of the arch through to the side of the buttresses and footings. (See illustrations below).

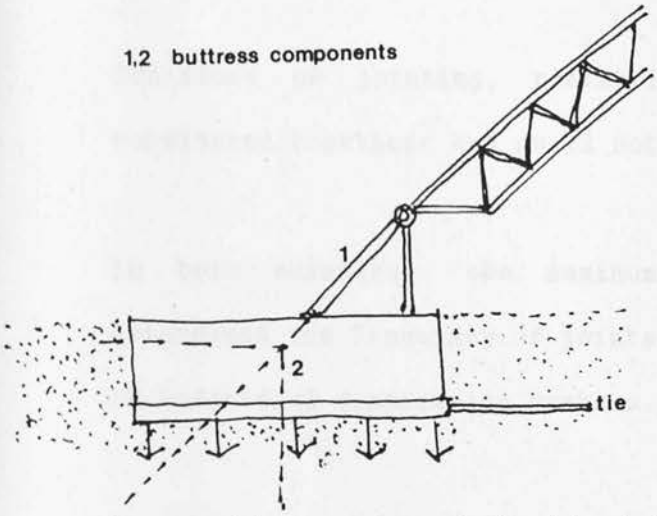


From an architectural point of view, it was considered important that the springing point for the arch trusses be located high enough to provide a side wall to the hall. This problem was solved by sitting the arches 2.8 m above ground level on a bipod frame set on an inverted tee rc footing, both of which comprised the buttress construction.



Horizontal thrusts of the arch were countered by a 75 mm diameter mild steel tie between opposite footings and the weight of the rc footings below grade. Architecturally, the bipod frame was visually compatible with the steel trussed arches, as all superstructure elements were kept in light structural framing. Keeping the supports slender allowed the wider openings at the side of the vault near ground level, improving visual links with the site surrounding.

The line of the inclined member of the bipod footing followed the line of thrust through the inverted tee-footing, and coincided with the centre of gravity of the pad footing.



The bipod frame receives the trussed arches and transmits the forces to the rc footing. The MS tie and the weight of the rc footing counters the horizontal thrusts of the arch

The vertical member of the bipod frame provided the stability for it to sit on the ground tee-footing before the planar trussed arches could be placed in position.

This expression is different in the case of the Waldstadion, where the curved line of the area was maintained through to the buttresses by conceiving the latter as "concrete fingers" which received the steel framed vault ends. This idea formed part of the final proposal after

the decision was made on a fixed ended arch, and thus the Waldstadion structural system could be described as one with fixed-ended arches arranged in a lamella fashion with part of it in steel and part of it in reinforced concrete (Figure 16).

The buttress made of reinforce concrete provided the weight necessary to resist the horizontal thrust of the arch and its inverted "Y" configuration was worked into the peripheral spaces (such as those of changing rooms) (Figure 14).

Concrete was also an appropriate material for the substructure of the intended turfing at the vault base edges - (This detail was presumed to express the form "growing" from the ground to further enhance its geometry, which was intended to harmonise with the site landscaping.

Decisions on jointing, prefabrication, erection and cladding are considered togetherr and could not be dominated by any single factor.

In both examples, the maximum desired size of cladding panel determined the frequency of joints and an aim to limit the slenderness of individual compression members.

In the case of the Liverpool Festival Hall, the selected cladding modules of 3 x 7 m led to the primary circumferential spacing of the Warren truss nodes. This gave eleven equal arc lengths of 3 m on the centre line of the upper boom, each side of the centre pin.

From this came the location of the longitudinal members connecting the arch trusses and the dome end.

The purlins supporting the cladding were acting principally as upper longitudinal members.

Joist sections were chosen as the principal upper and lower booms. this gave the advantage of clarity of line whilst minimising the bulk and making connection easy. the joist section could also be used as a runway for an access cradle for servicing purposes. (Ref pg 6, OAP Journal, October 1984).

In the Waldstadion example, brief consideration was given to the possibility of structural units with pre-attached cladding panels, even using composite action between frame and skin.

However, this was not parallel with the intention of interchangeable panels (similar to the system used in the Sainsbury Arts Centre designed by Norman Foster Associates), and the idea was not pursued.

In order to facilitate water-proofing and drainage, most cladding systems favoured their own sub-frame which was one separate from those of secondary structural members. In the case of the Waldstadion there would have been a wasteful duplication of structural members at the mating edges with the idea of pre-attached panel.

Not all the patterns of repeatable units reconstituted the complete roof pattern on assembly. Variation on a particular unit which permitted flexibility in assembly was also desired. Other practical considerations included the minimisation in the number of crane lifts with units of larger size and the avoidance of high shear forces in the jointing elements. (Single bolt joints experienced high shear in addition to the axial force transmitted).

The fold lines that gave the roof its segmentally curved shape, occurred at the edges of the hexagonal cells rather than across them in order to simplify the cladding panels (Figure 8).

This meant that for certain configurations of units, angle changes in their main chords would have occurred at their "field joints" rather than at their shop joints, producing in turn, chord force eccentricity within any of the field joints considered (Figure 17).

The erection method for the Waldstadion consisted of a travelling addition of successive complete arches of repeated units <u>against</u> an initial gable arch of specially braced units.

The erection method required two provisions:

i) The introduction of self-weight into each new completed arch.

ii) A system of moveable bracing to hold temporarily unequilibrated forces in the newly added chord members. Only upon completion of the roof do the longitudinal chords act to contain the spreading effect of the main transverse chords.

The simplest erection sequence would be to progress <u>transversely</u> from opposite abutments simultaneously; propping the full length of the roof twice at each advancing front before reaching the crown. Only a third or half of the full length of the roof need be erected at one time to allow for completion of the abutments.

The design implication in joint design was that the joints were to allow sub-assemblies of units in the partially erected roof from sliding vertically alongside adjacent assemblies, and was subsequently

abandoned in this respect (Figure 18).

In the case of the Liverpool Festival Hall, conventional in-situ concrete techniques were used to cast the heavy foundation bases for the arch systems. The individual bases were tied together both longitudinally and laterally by 75 mm tensioned tie rods (Figure 19) which themselves were subsequently encased in concrete.

Foundations for the dome ends comprised semi-circular rc ring beams, again cast in-situ by conventional techniques.

During the later stages of this work, delivery of the part-prefabricated steelwork members began. Due to carriage restraints (of 22 m length) individual trusses were delivered in two sections which were welded together on site prior to their location by pin to the bipod springings in readiness for the final lift of the crown and insertion of the central pin to form the completed arch section. This was followed by the installation of purlin members.

Construction of the dome and steelwork started with the upper semi-circular stiffening girder which was supported by centering until dome ribs were subsequently placed.

Services Consideration

Ventilation ducts which could be located within the zone occupied by the structure assisted in minimising total structural depth, and freeing headroom space.

The Warren truss webbing had the special attribute of allowing ventilation ducts to pass through it along offset lines of the same grid.

The advantage influenced its selection as the final configuration adopted for the truss webbing, in the Waldstadion proposal.

FRANKFURT ATHLETIC STADIUM

List of translations for German annotations

1 Sonnenschutzlamellen - sunshading elements.

2 Kombiniertes Brandlueftungs - lueftungs und Tageslichtelement - combined fresh air, mechanical ventilation and daylight system.

3 Ausblendraster - diffuser.

4 Kunstlicht - artificial lighting.

5 Edeistahideckung - stainless steel cladding.

6 Daemmung - insulation.

7 Akustikinnenschale - acoustic panel.

8 Metallunterkonstruktion - metal substructure.

9 Stahironhrkonstruktion - steel tube construction.

10 Lamellen fuer sommerlichen Weermeschutz - blinds for insulation in summer to get warm air (out/in?).

11 Blendschutzlamellen - blind/glare.

12 Einfachverglasung - single glazing.

13 Konvektionshelzung - convection heating.

14 Zu-und Abluft Nebenraueme - fresh air exhaust for secondary rooms.

15 Wasser und Elektroversorgungsschacht - shaft for services supply.

16 Massivtribuene - main grandstand.

17 Fluchtkorridor - escape corridor.

18 Teleskoptribuene fuer Wettkaempfe - telescopic grandstand seats.

19 Trainingssprintbahn - training track (sprints)

20 Begruente Anboeschung - turfing.

21 Wasserdichte Daemmung - water-proofing.

22 Betonwand - concrete wall.

23 Entwaesserungsrinne - drain (gutter detail?).

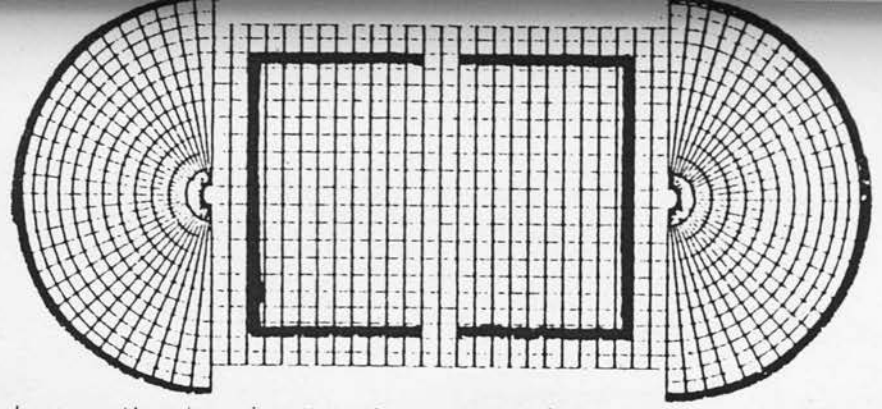
24 Hallenzuluft - fresh air supply for hall.

25 Glasbetonoberlicht - glass blocks.

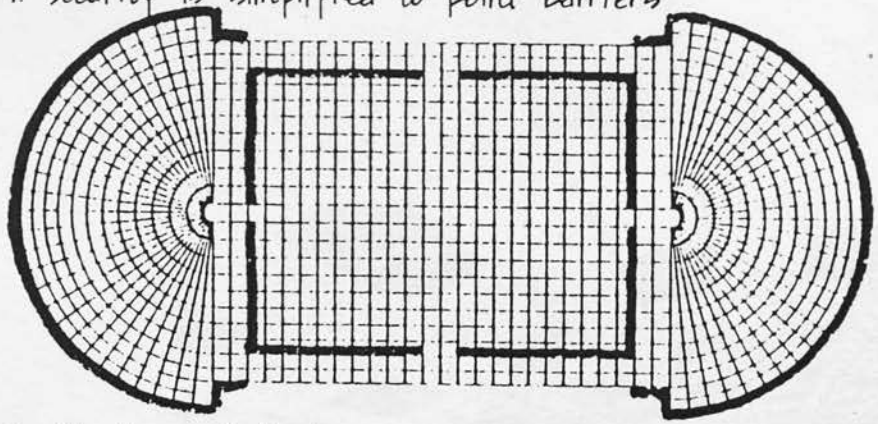
26 Betonwiderlager - concrete buttress (literally, anti-thrust).

27 Isollerverglasung - thermal glazing.

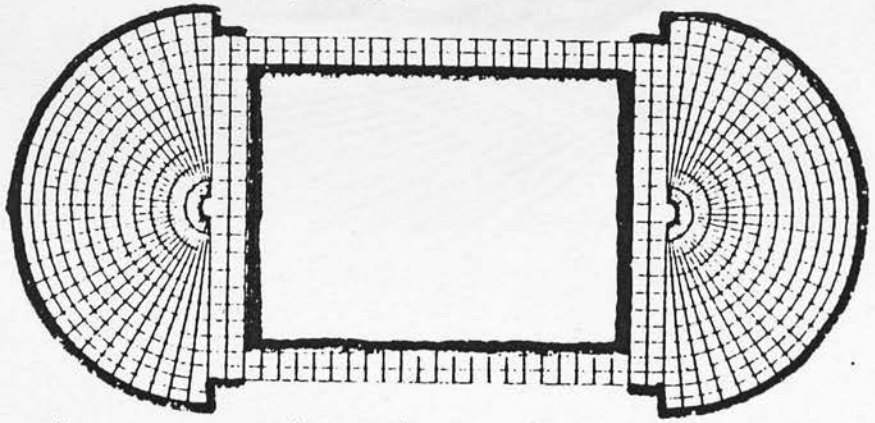
28 Strahiheizungspaneel - solar panel.



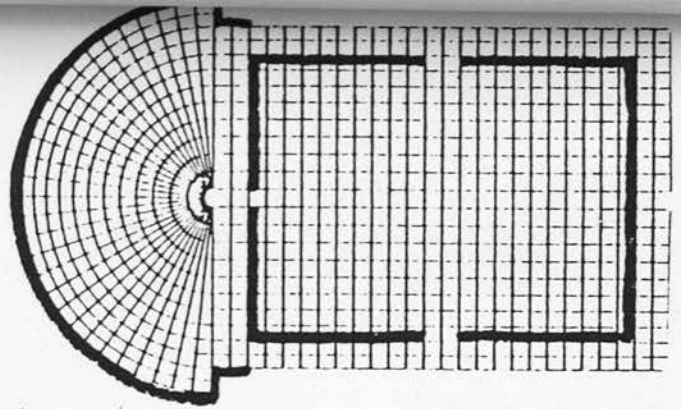
1. security is simplified to point barriers



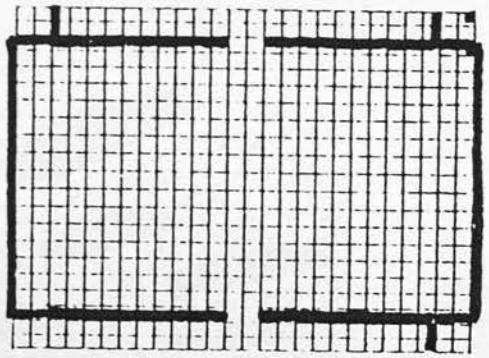
2. all three halls in use



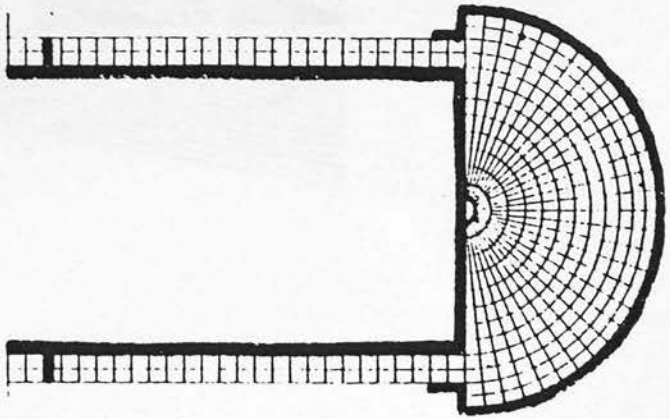
3. the two end halls in use & linked by foyers



4. two adjacent halls in use.



5. Only the centre hall.



6. Only an end hall.

FIG 1A

FESTIVAL HALLS ALL INTERCONNECT IN ANY COMBINATION

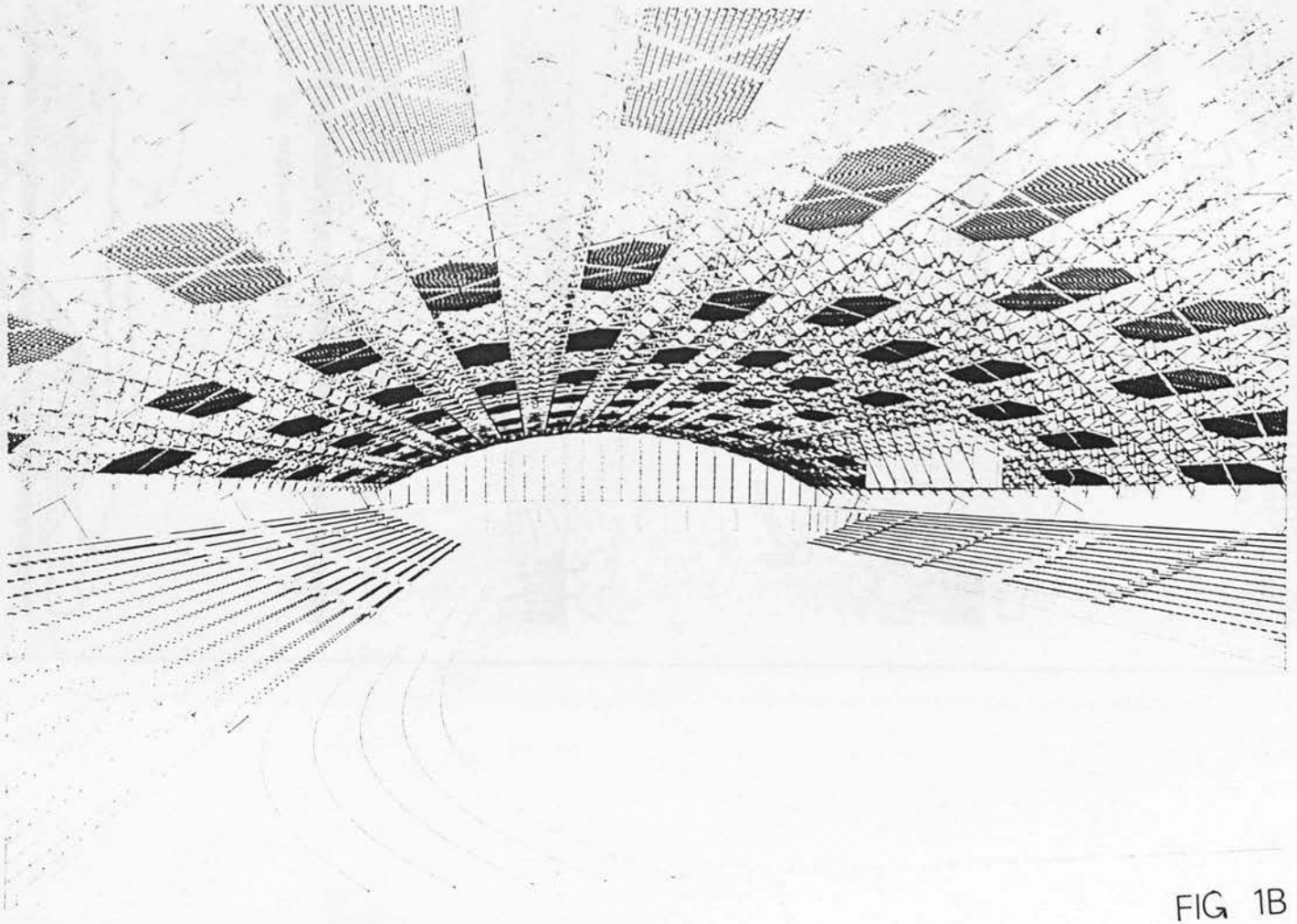
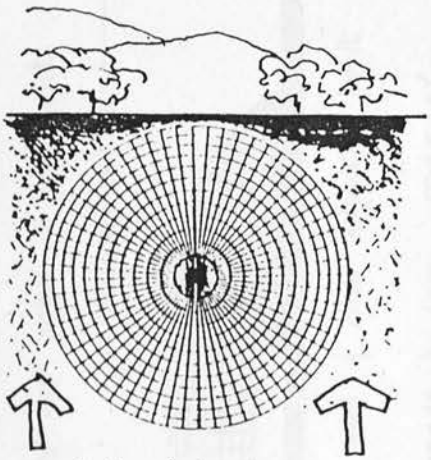
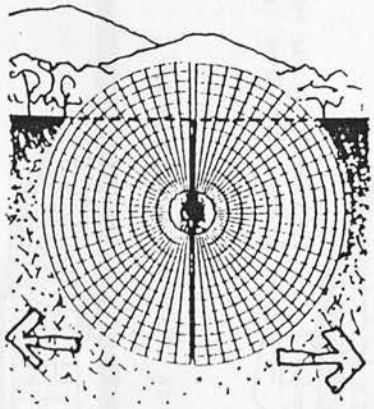


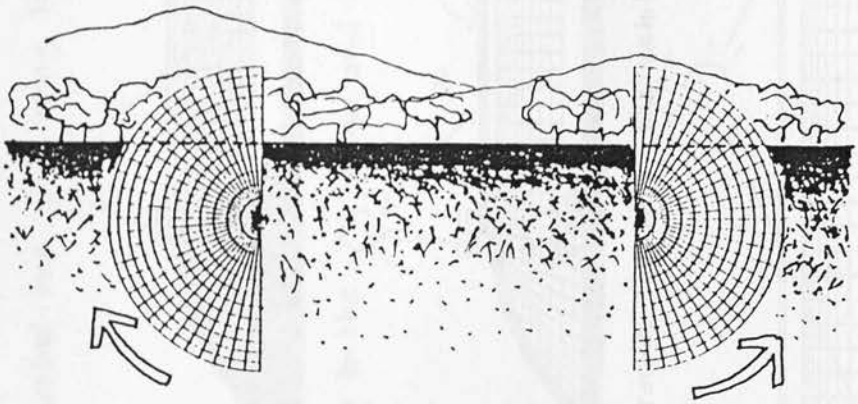
FIG 1B



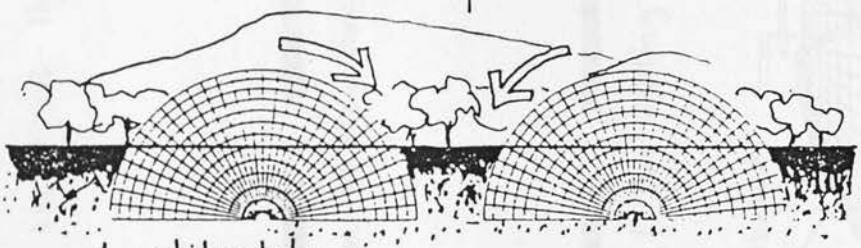
1. a hollow hidden sphere



2.....emerges

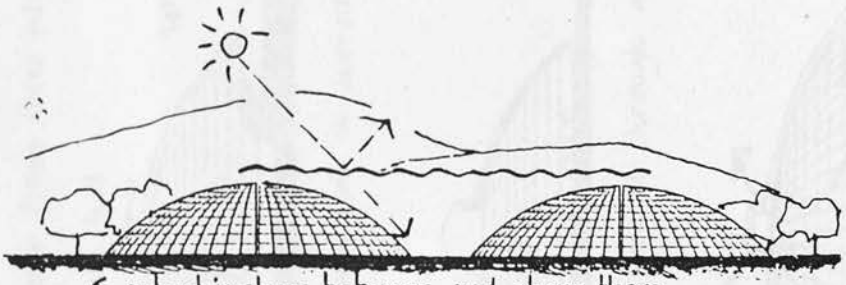


3.....divides into two hemispheres

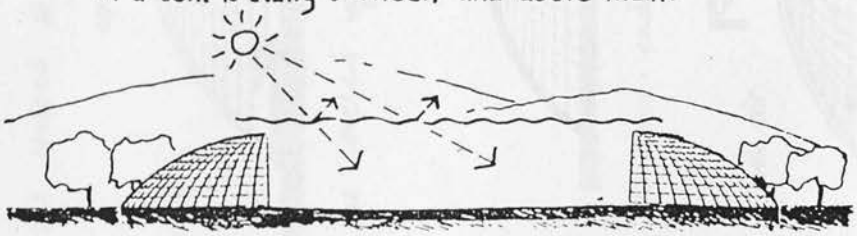


4.....which rotate,

5...to form two man-made hills.



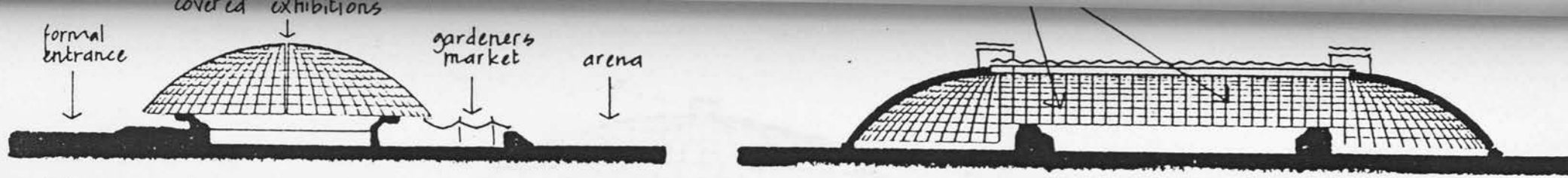
6. a tent is slung between and above them.



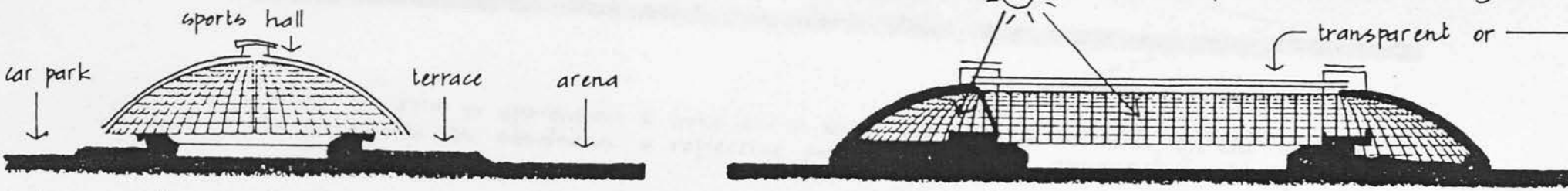
7. the inner halves disappear giving two caves & a garden.

The form emerges

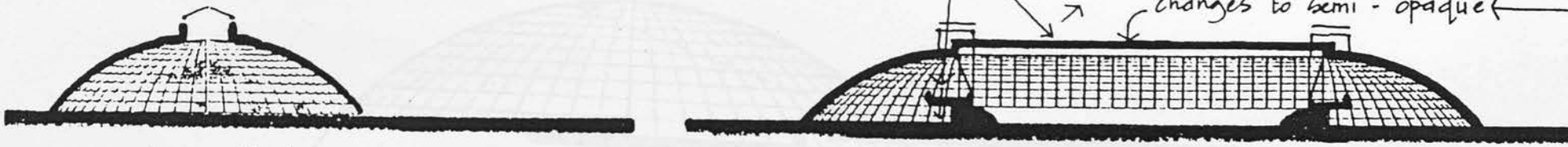
Fig 2A



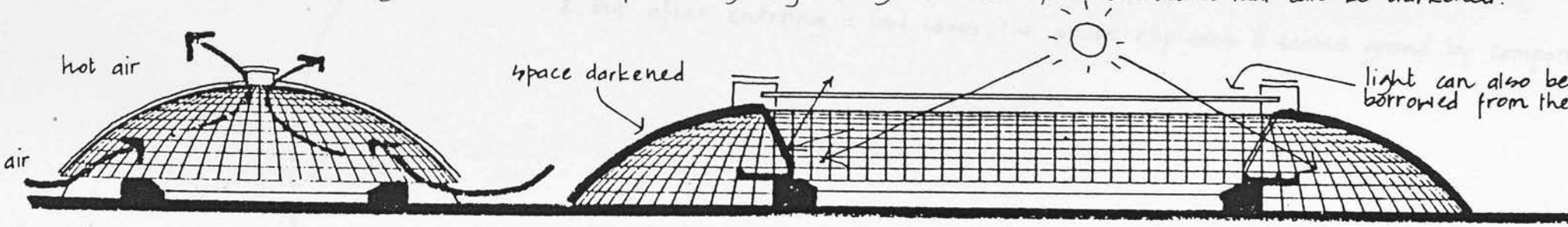
1. the covered arena forms three internal spaces for the festival - the central one being totally top open to the light.



2. the covered sports hall is surrounded by seats - relates directly to the arena and has pool & club adjacent to it

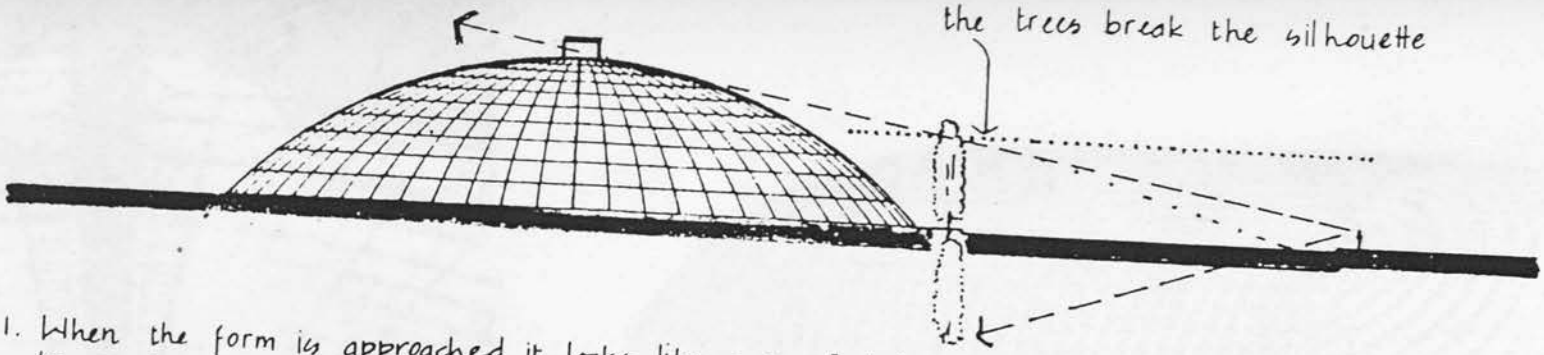


3. the cave - ends are lit through an oculus with further lighting through the skin, the exhibition hall can be darkened.

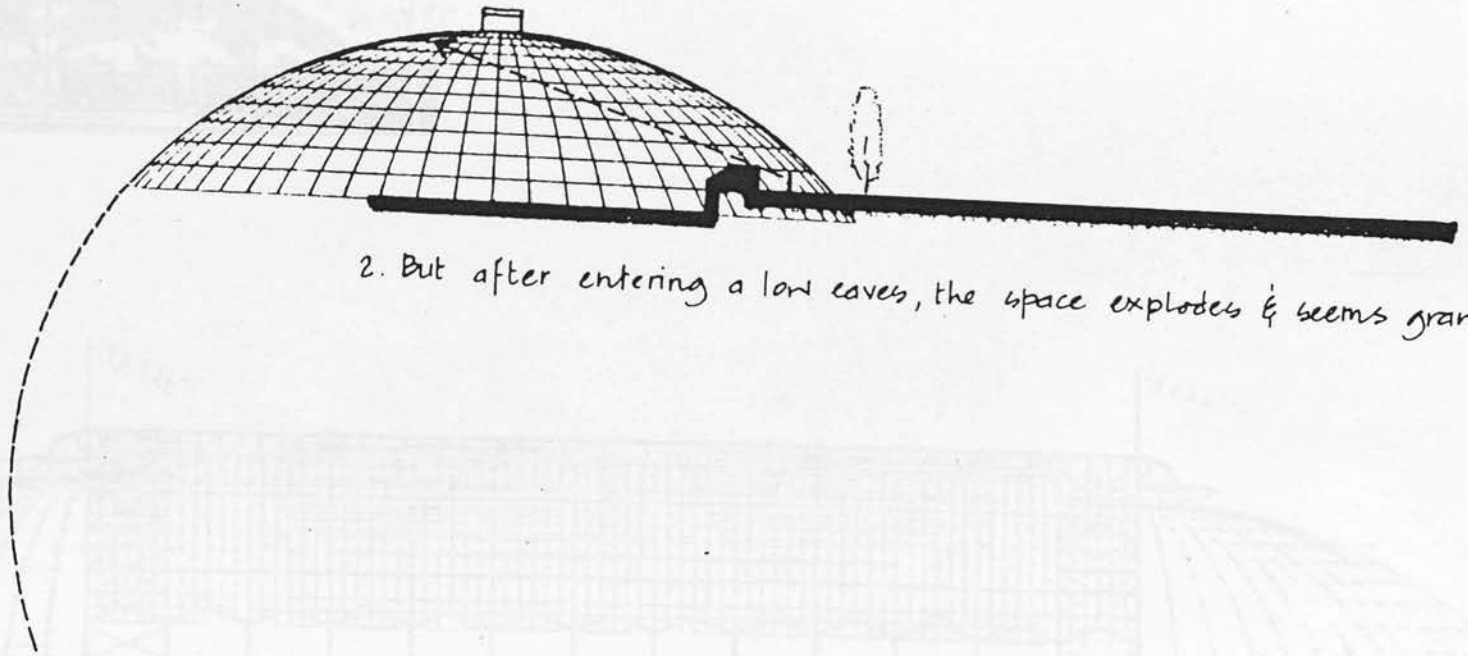


4. ventilation is naturally achieved - from bottom edge to centre, the side caves can be darkened with canvas.

FIG 2B THE GROUND FORMS AN ARENA WITHIN THE ENCLOSU

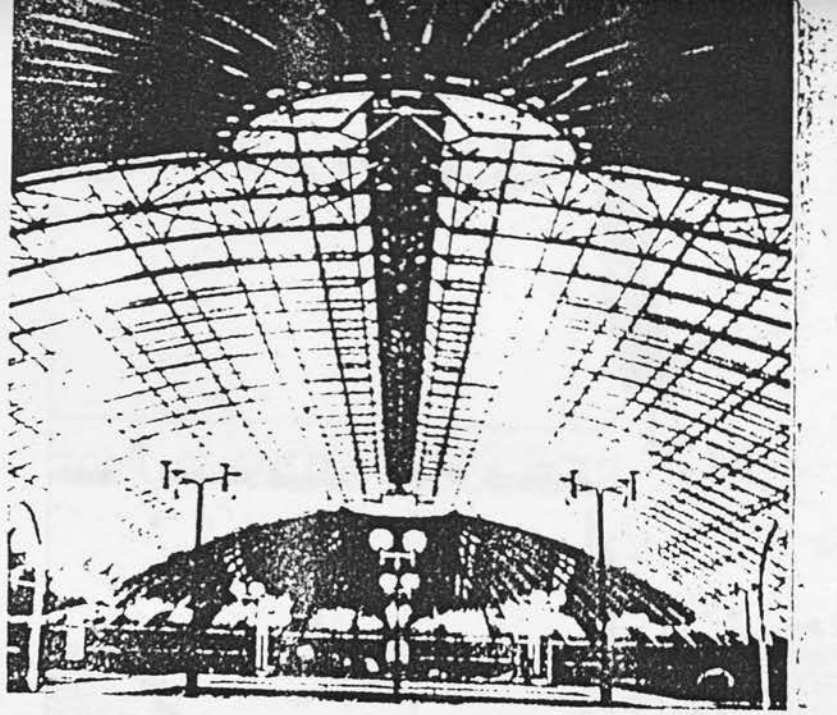


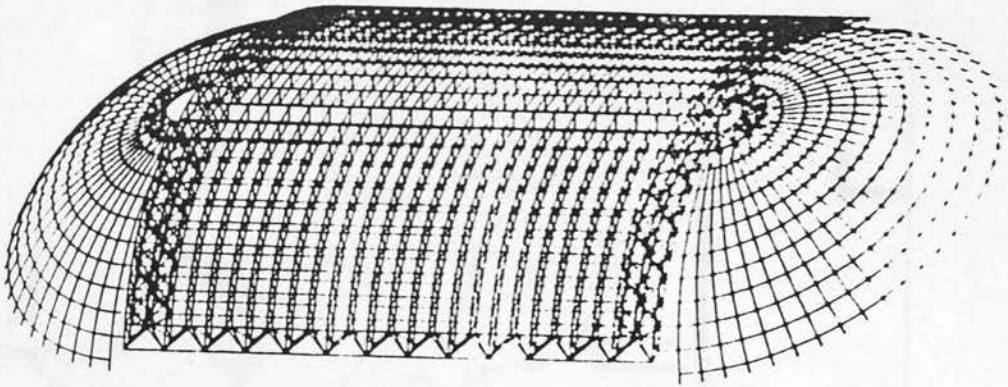
1. When the form is approached it looks like a small hill - green verticals cut the skyline to emphasise its smallness, a reflective pool increases the apparent height of the green vertical



2. But after entering a low eaves, the space explodes & seems grand by compari

FIG 3. THE FORM IS APPROACHED





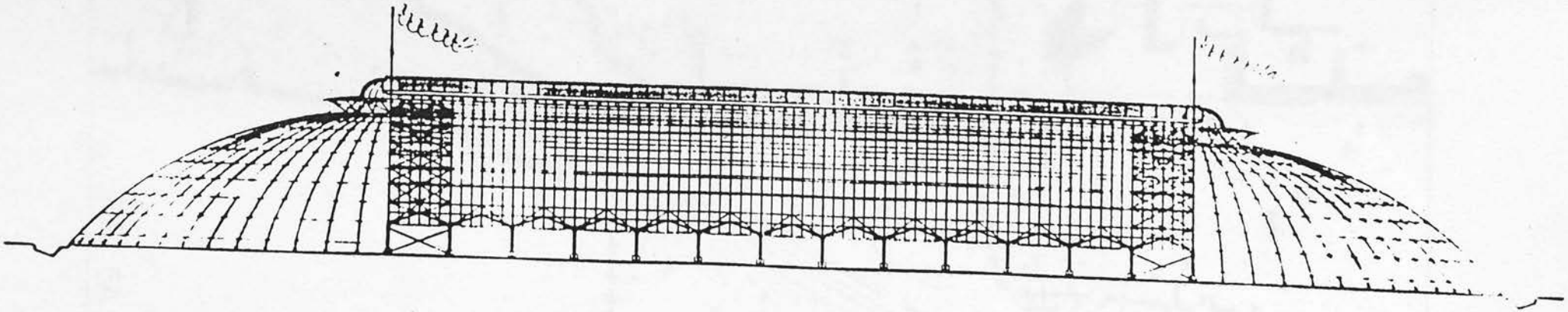


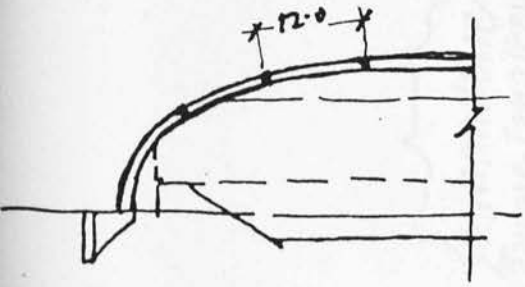
FIG 4

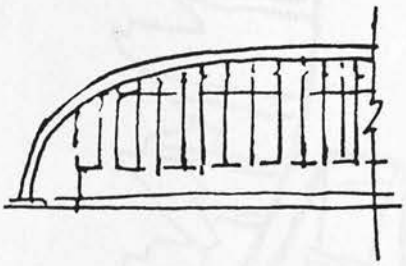
Lechnerbäude in Frankfurt Höchstadt
Foster Associates April 1982
Legende 1:500

FIG 5A

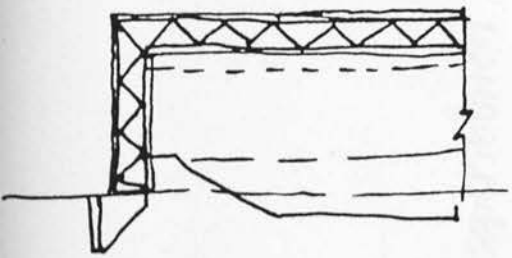
MOST EXPENSIVE

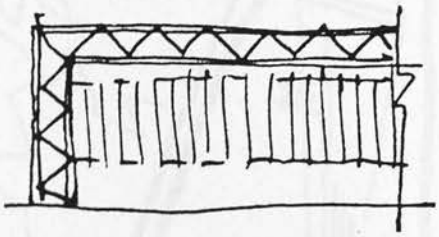
FIG 5B



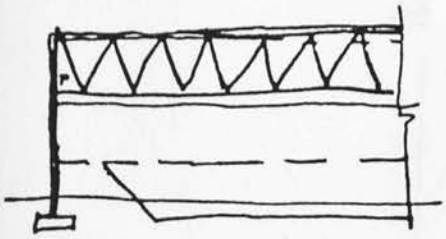


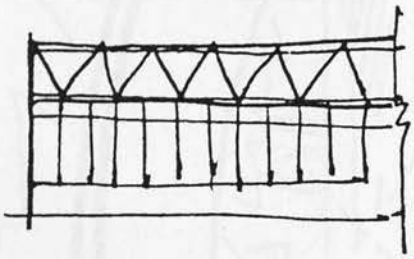
1. CIRCULAR STEEL PLATE MONOCOQUE WITH DIAPHRAGM SEPARATING TWO STRESSED SURFACES



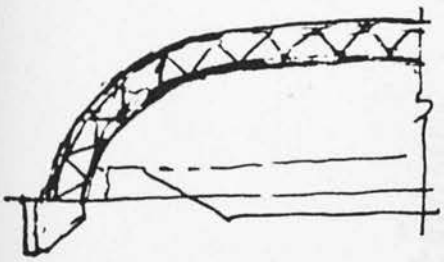


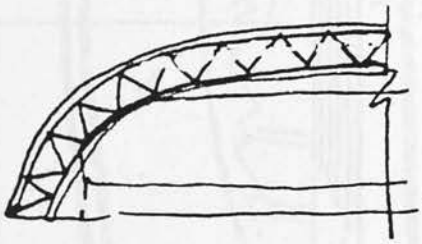
2. RECTANGULAR TRUSSED PORTAL



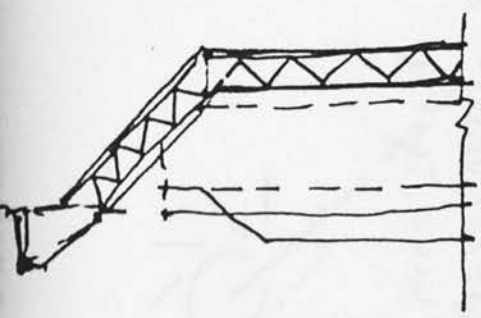


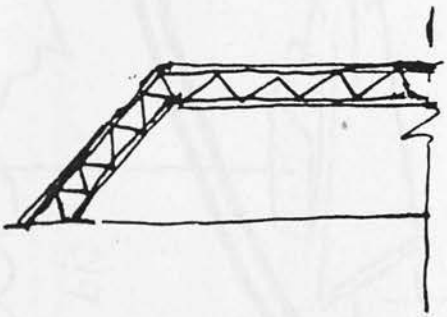
3. SIMPLY SUPPORTED TRUSS



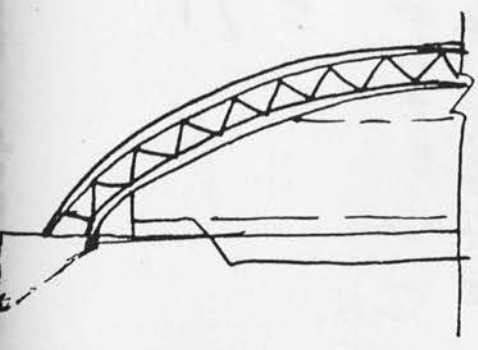


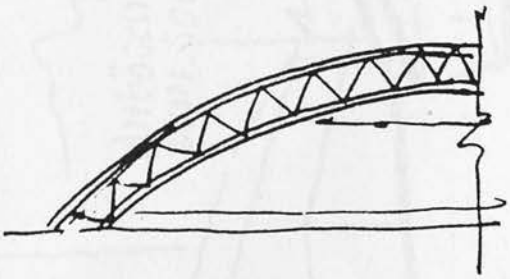
4. THE ELLIPTICAL ARCH





5 - THE CRANKED PORTAL





6. CIRCULAR TRUSSED ARCH

LEAST EXPENSIVE

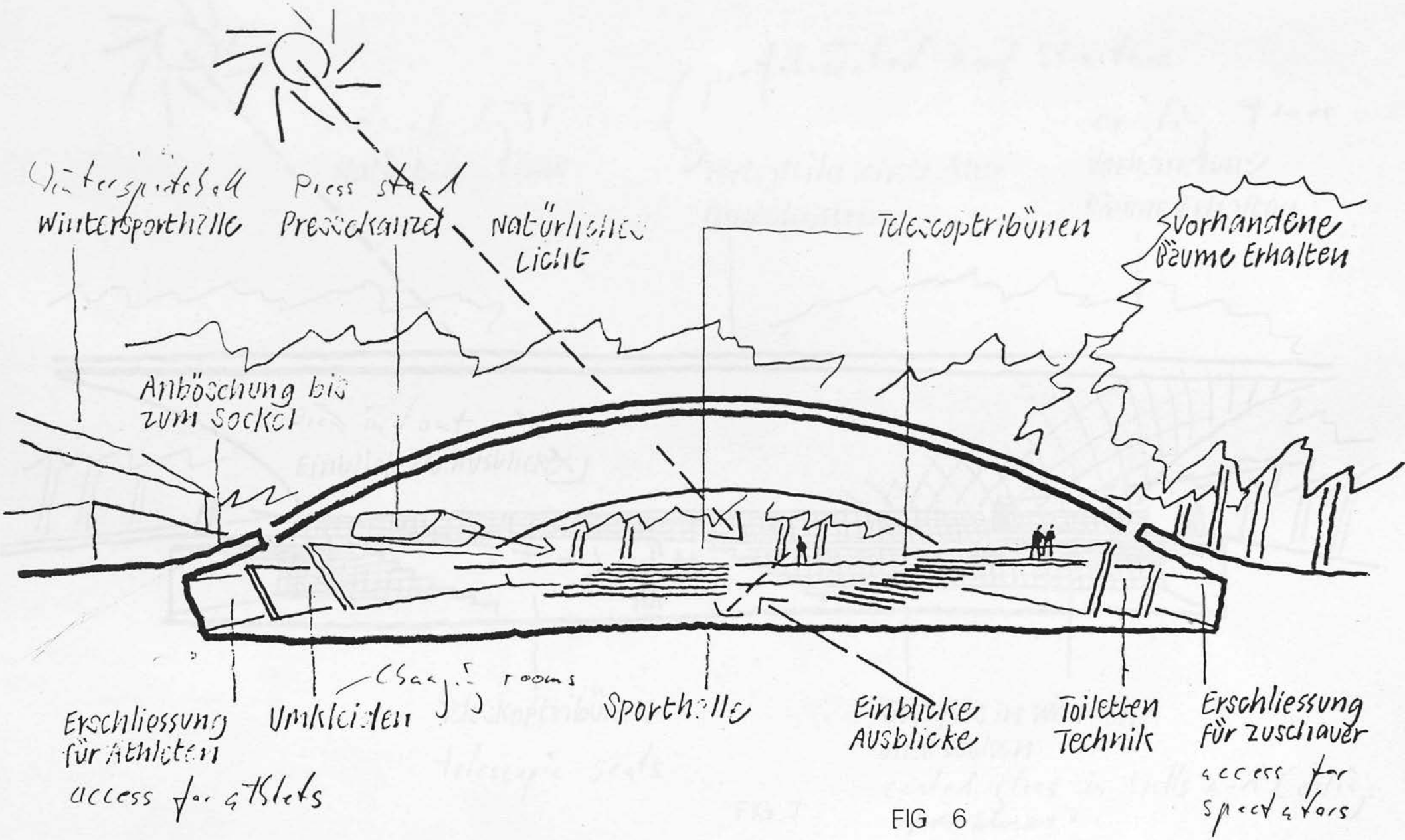

Wintersporthalle
Pressekanzel
Natürliches Licht
Teleskoptribünen
Vorhandene Bäume Erhalten
Anböschung bis zum Sockel
Erschliessung für Athleten
access for athletes
Umkleiden
Sporthalle
Einblicke Ausblicke
Toiletten Technik
Erschliessung für Zuschauer
access for spectators

FIG 6

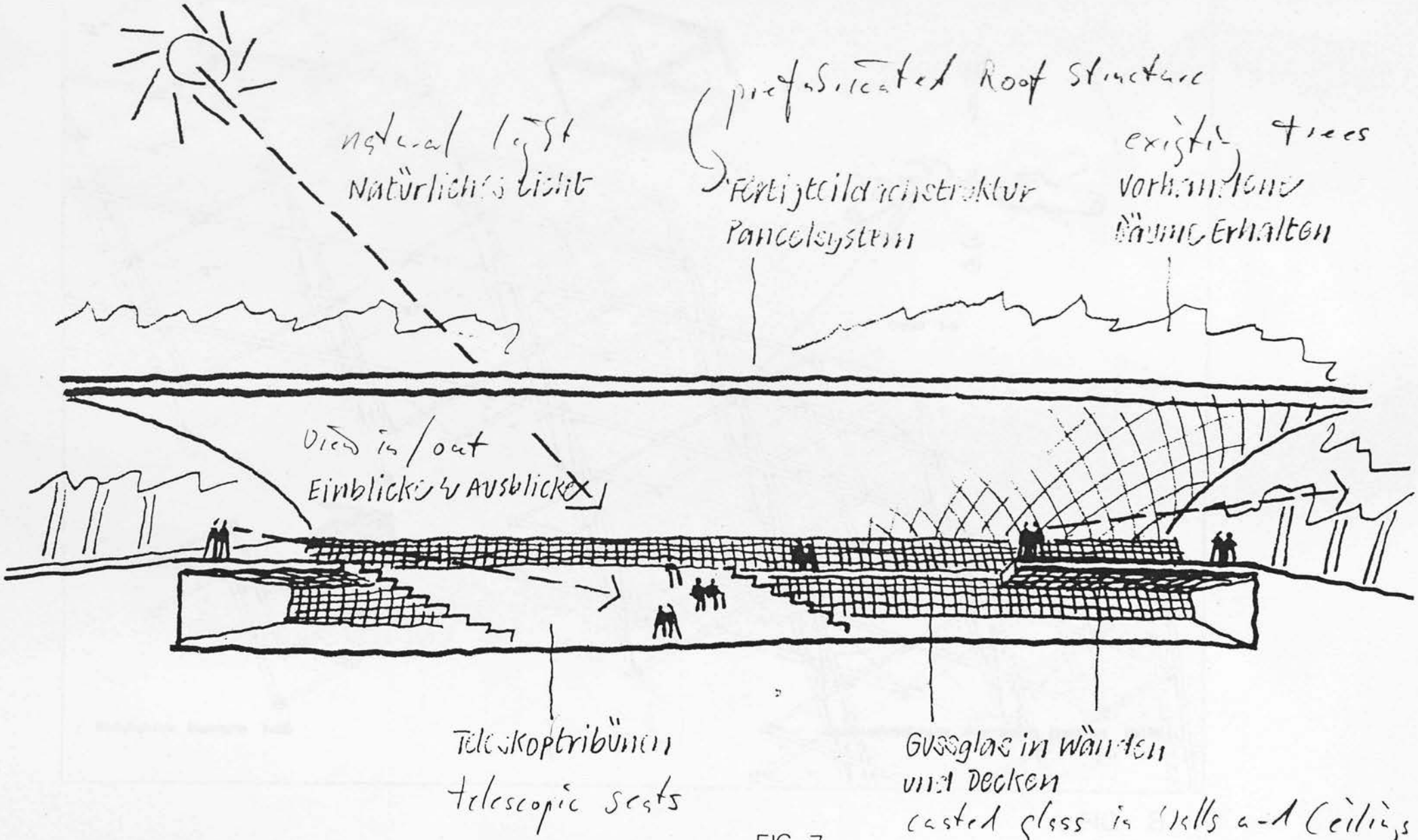

natural light
Natürliches Licht
prefabricated Roof Structure
Fertigteilkonstruktur
Panelsystem
existing trees
vorhandene
Bäume Erhalten
View in/out
Einblicke & Ausblicke
Teleskoptribünen
telescopic seats
Gussglas in Wänden
und Decken
casted glass in Walls and Ceilings
Pavilions?

FIG 7

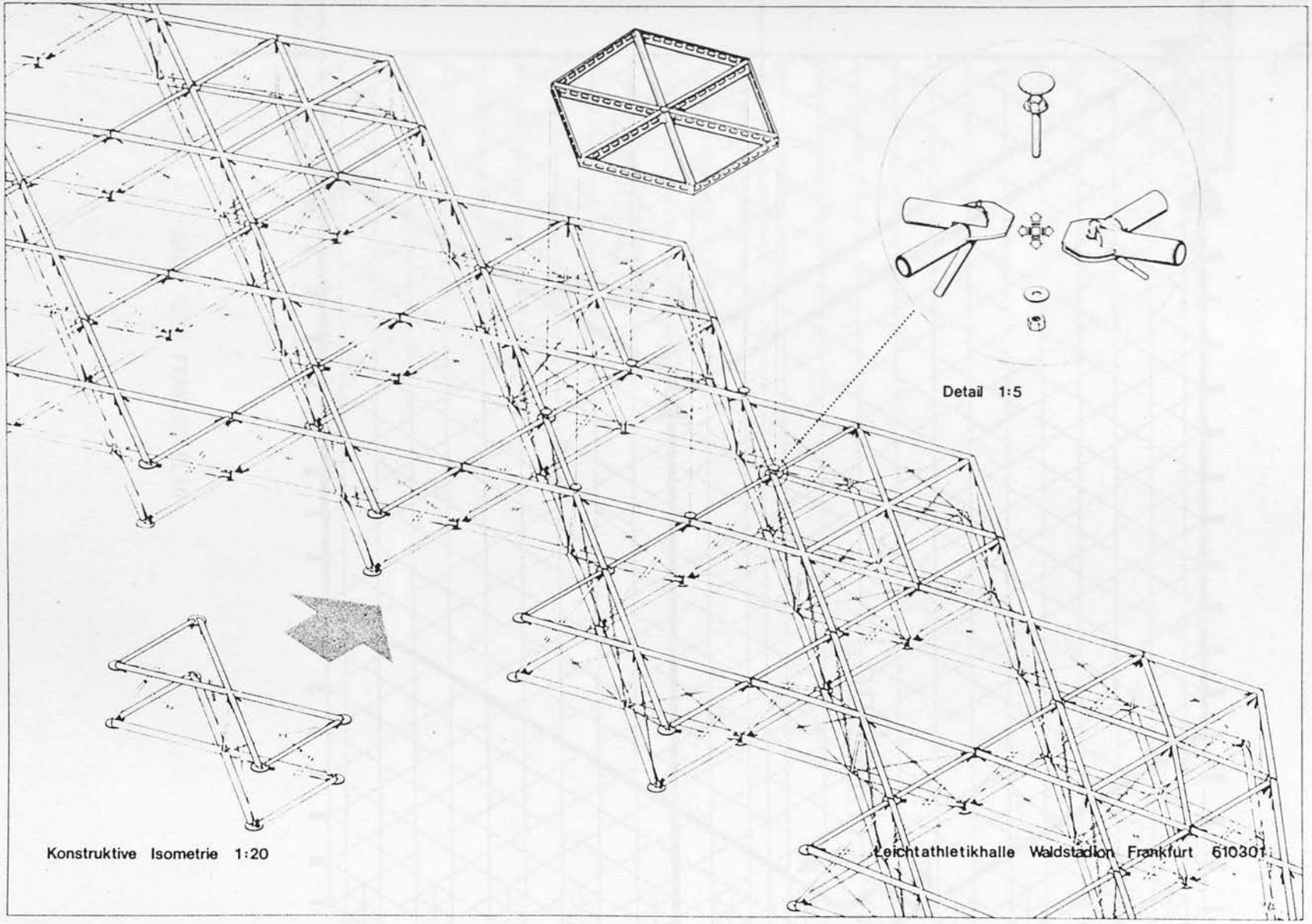

Detail 1:5
Konstruktive Isometrie 1:20
Leichtathletikhalle Waldstadion Frankfurt 610301

FIG 8

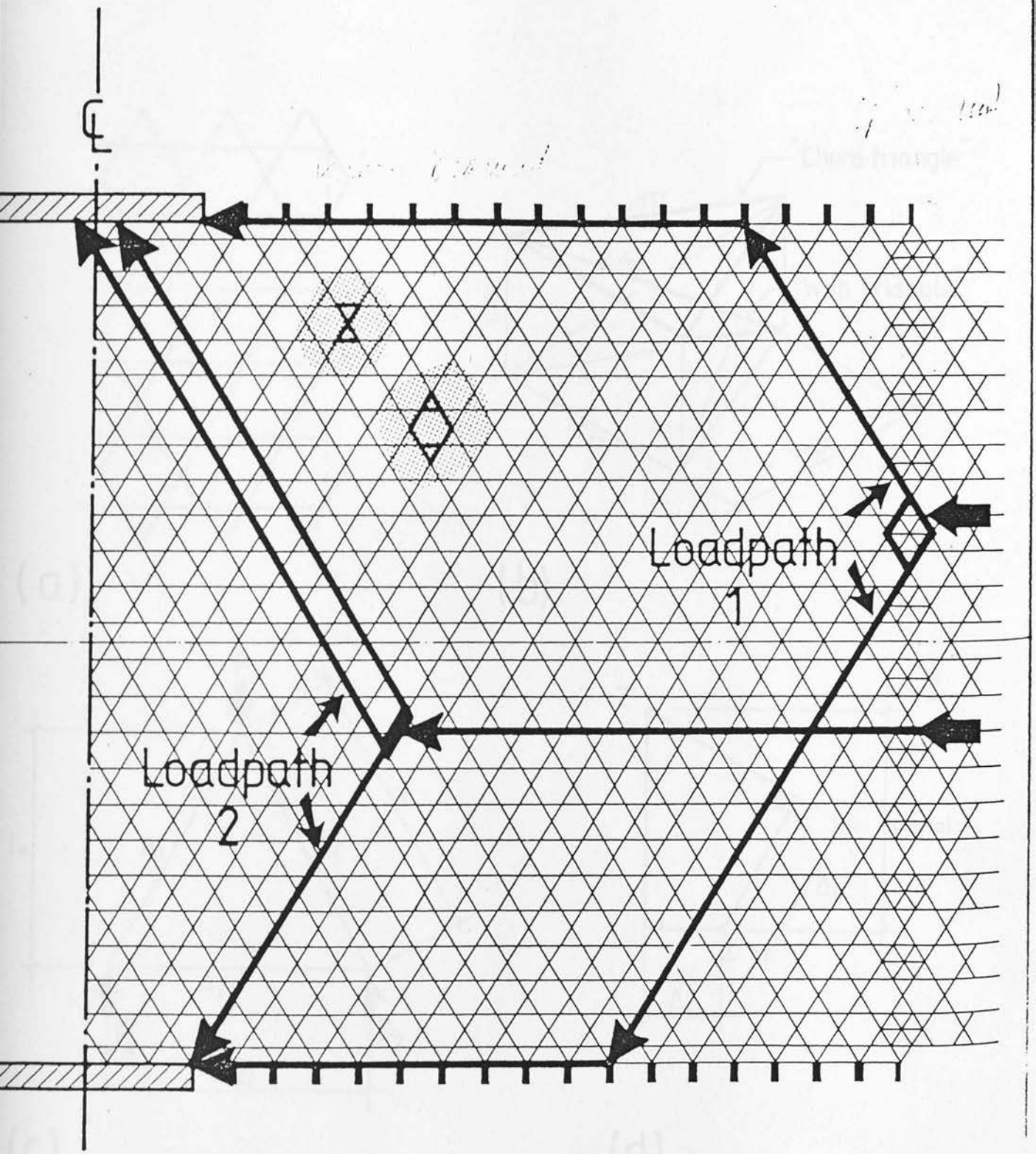
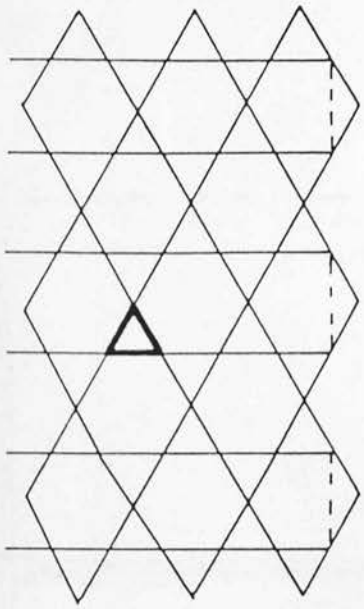
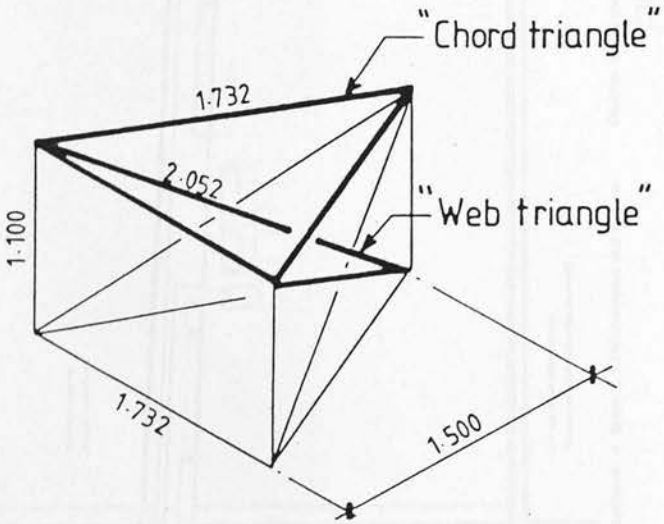


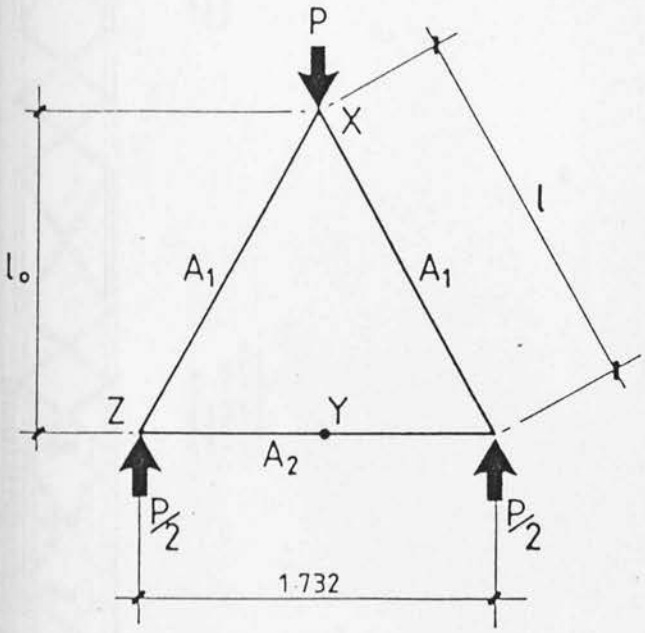
FIG. 9 HALF ROOF FRAMING PLAN



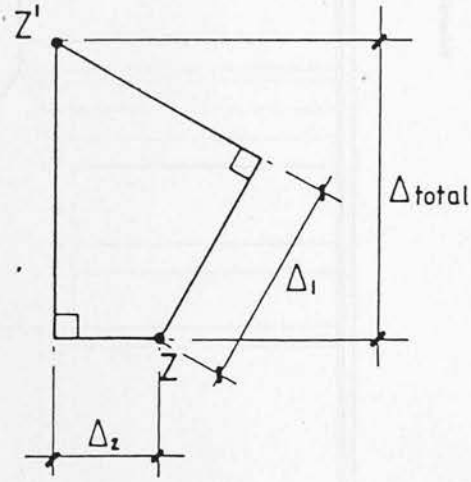
(a).



(b).

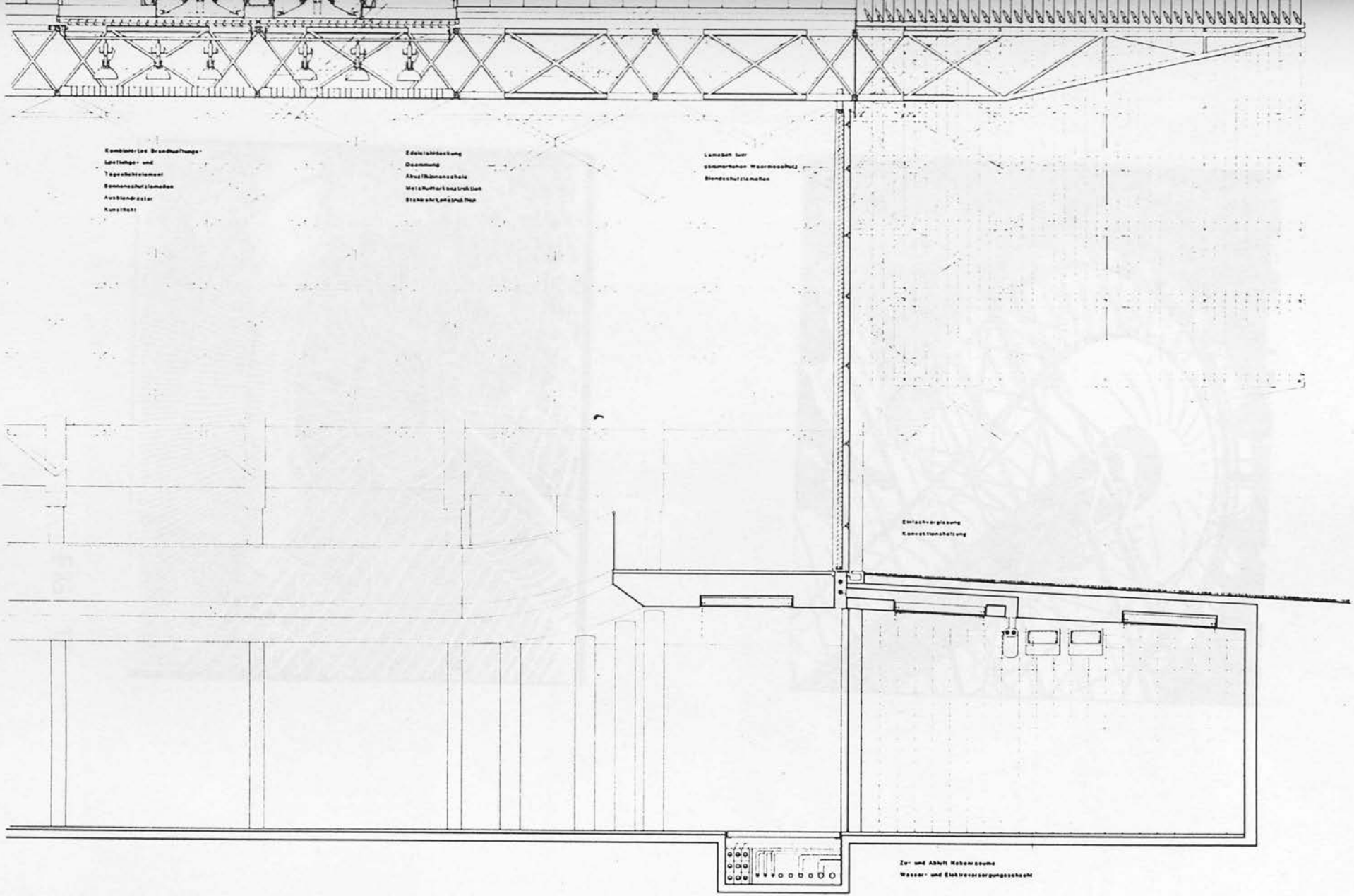


(c).



(d).

FIG. 10 REDUCTION FROM 3 TO 2 DIMENSIONAL MODEL FOR TRANSVERSE ANALYSIS

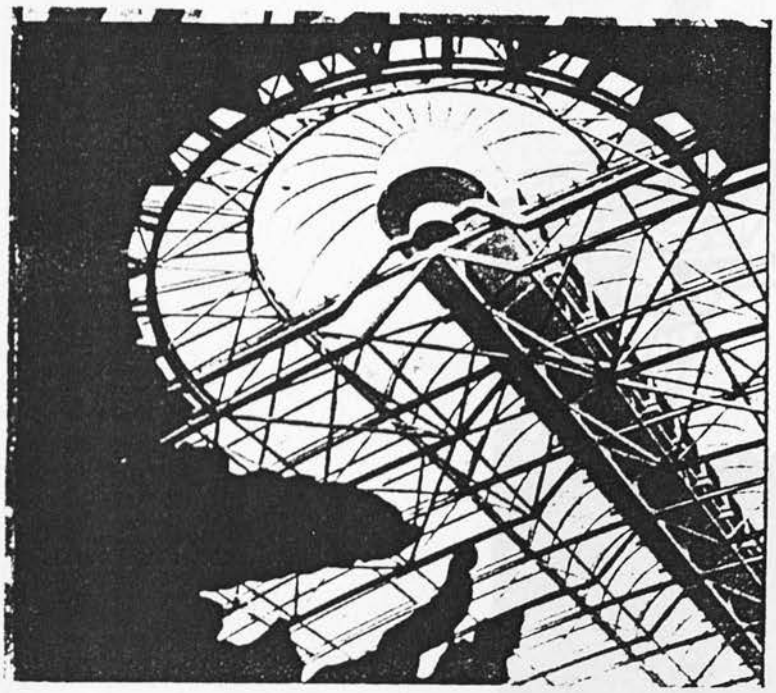


Leichtathletikhalle im Frankfurter Waldstadion

Architektengemeinschaft Foster Associates - Braun & Schlockermann und Partner

Oktober 86 Laengsschnitt M 1:20

FIG 11



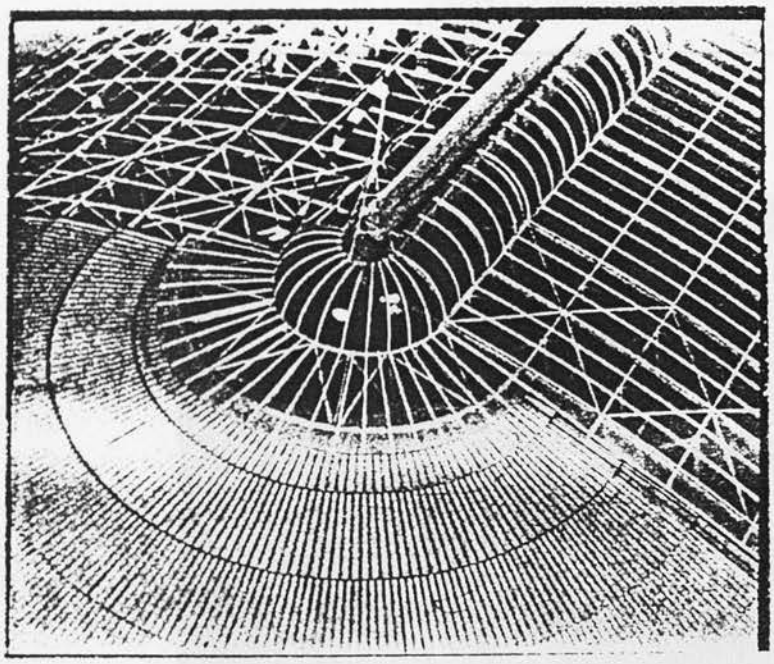


FIG 12

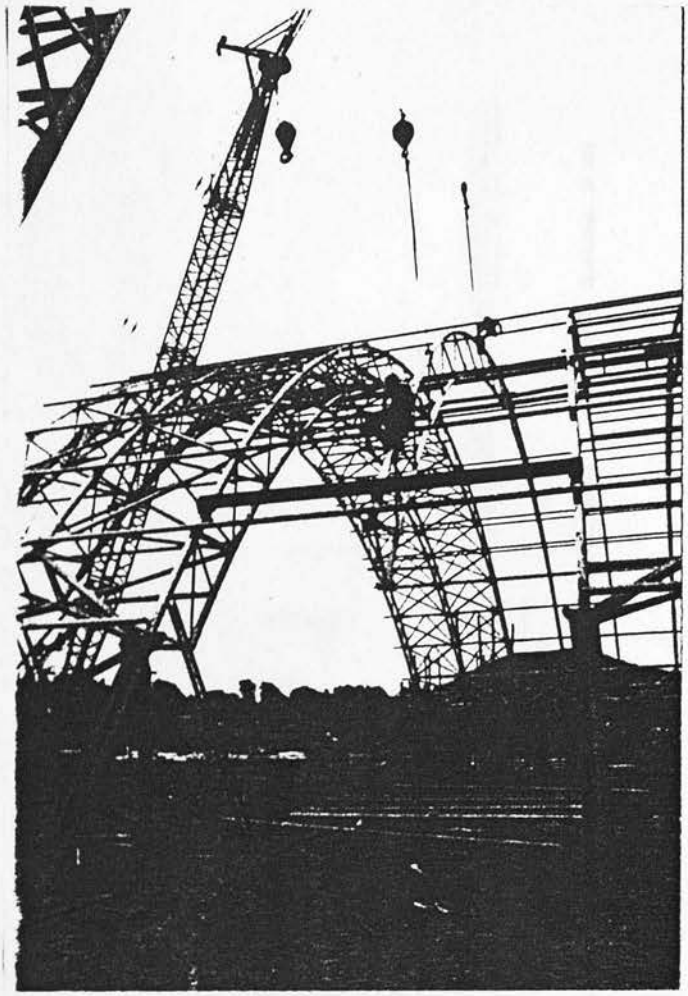
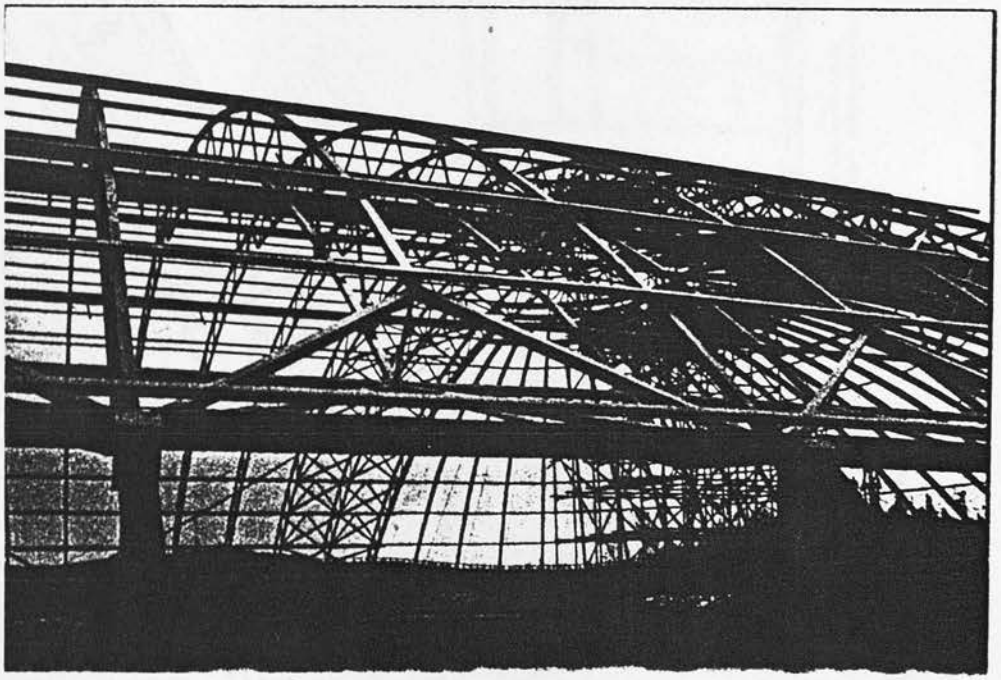
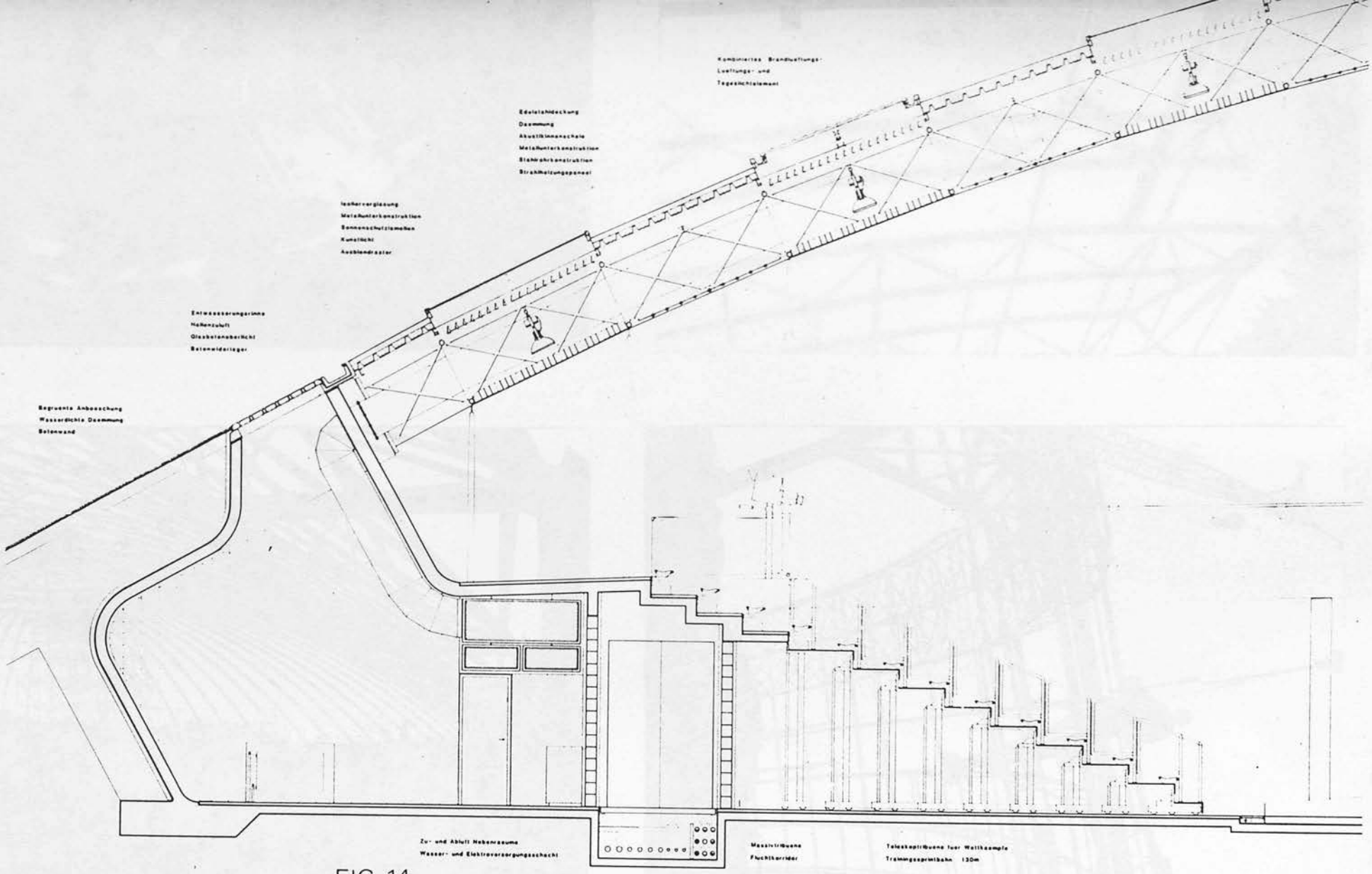


FIG 13



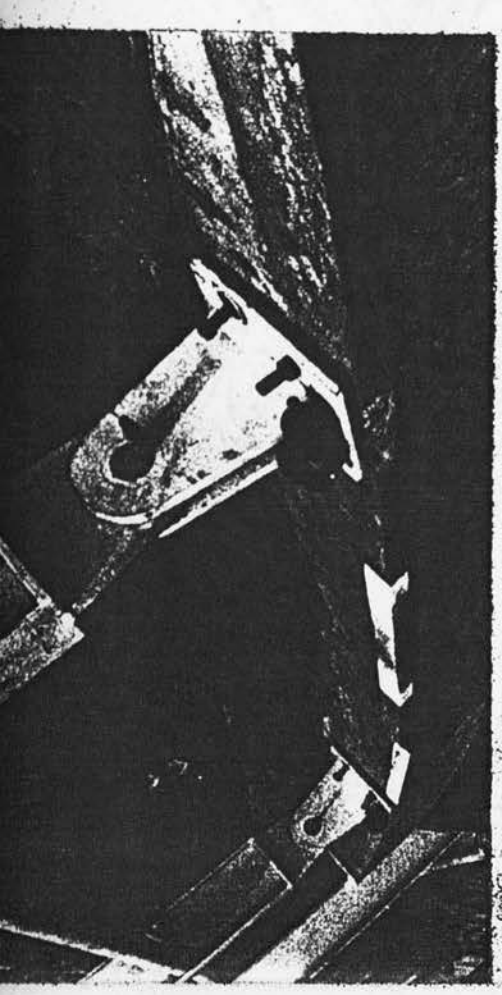


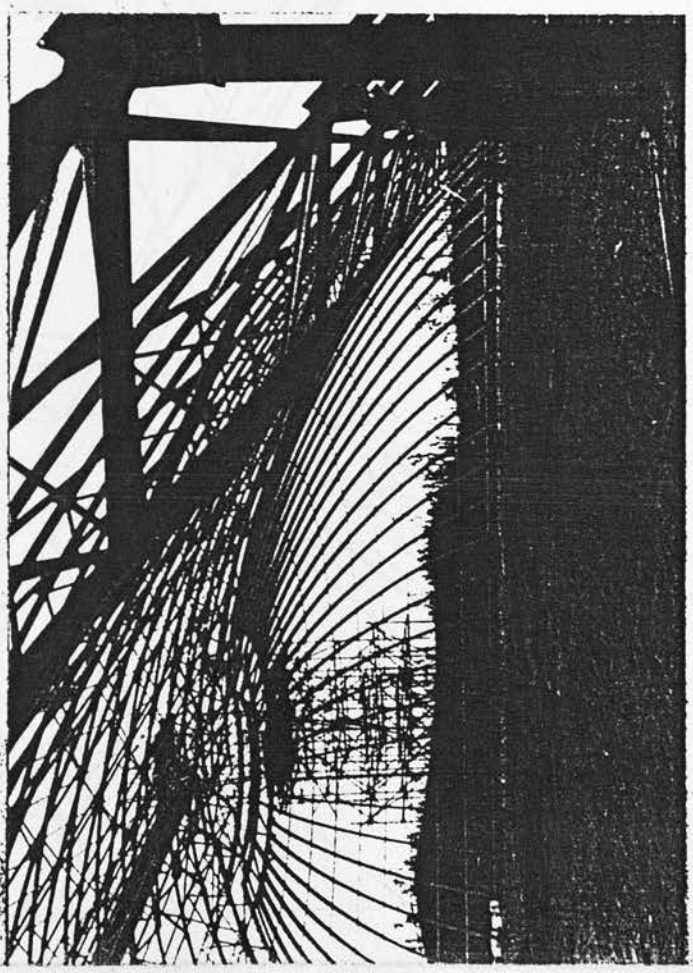
Leichtathletikhalle im Frankfurter Waldstadion

FIG 14

Architektengemeinschaft Foster Associates - Braun & Schlockermann und Partner Oktober 86 Querschnitt M 1:20

FIG 15





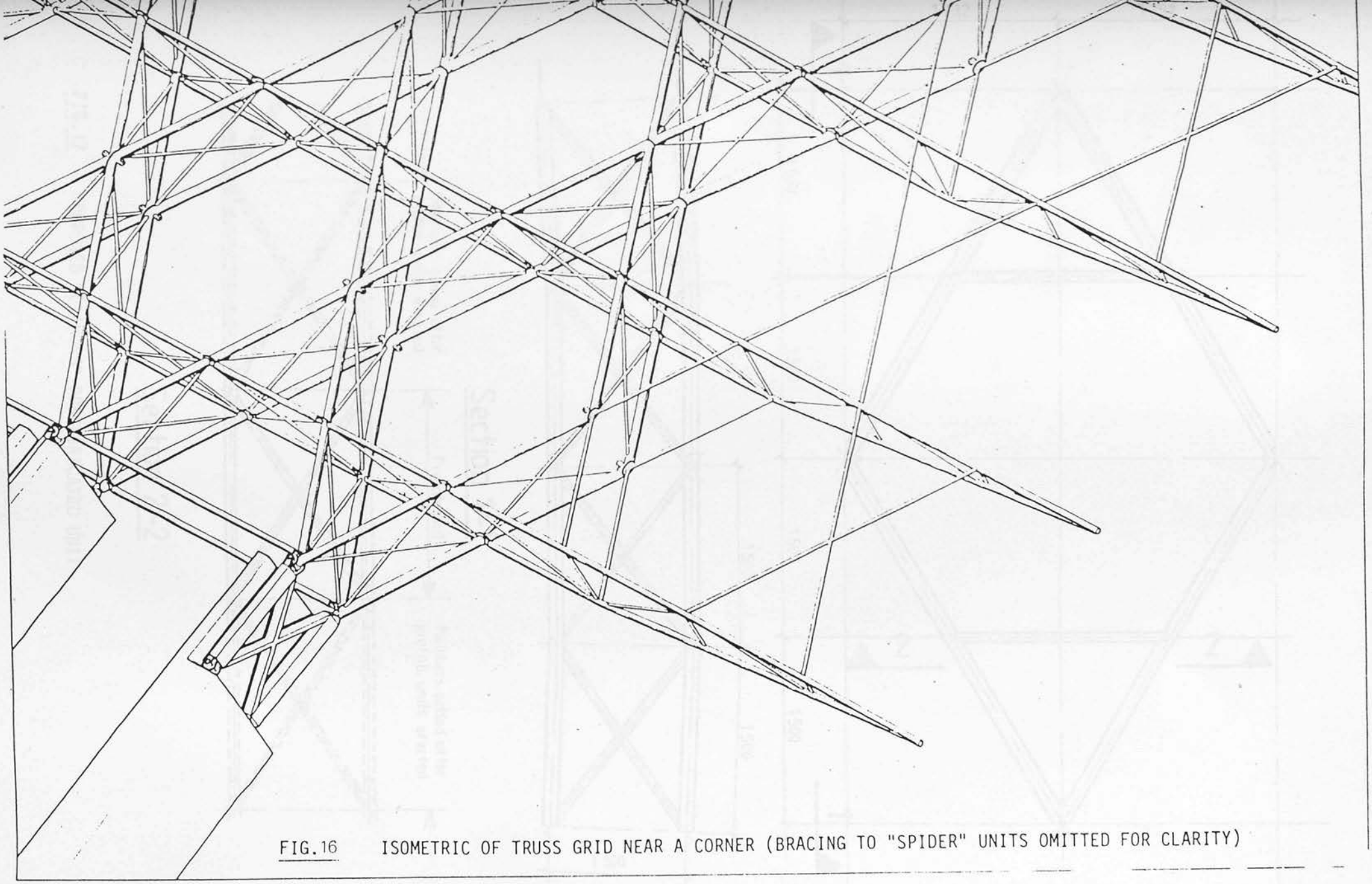
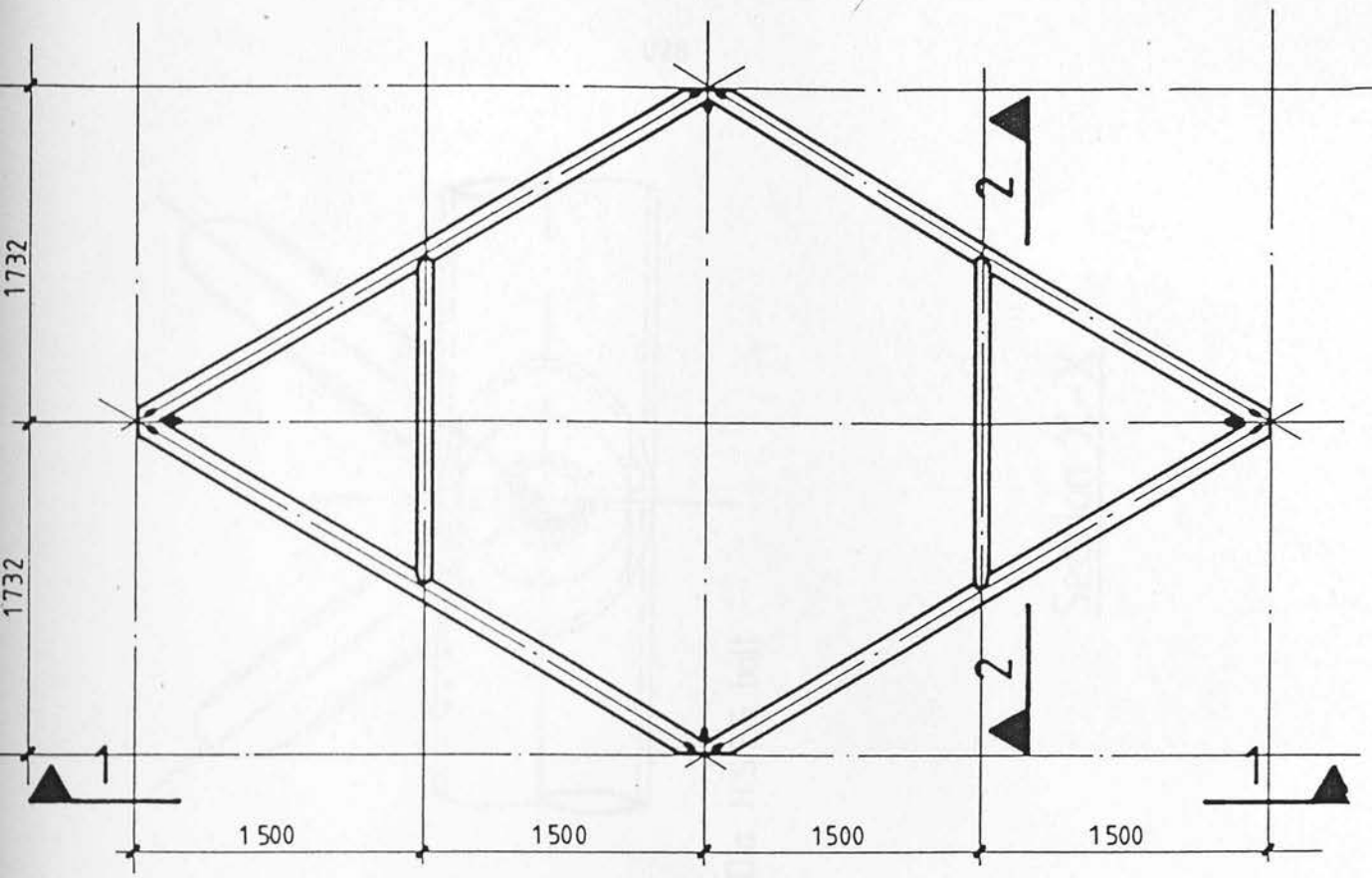
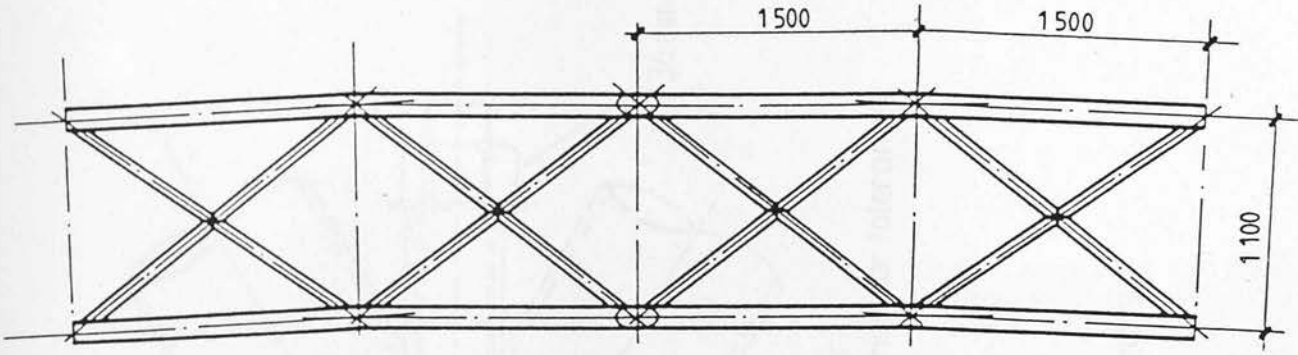
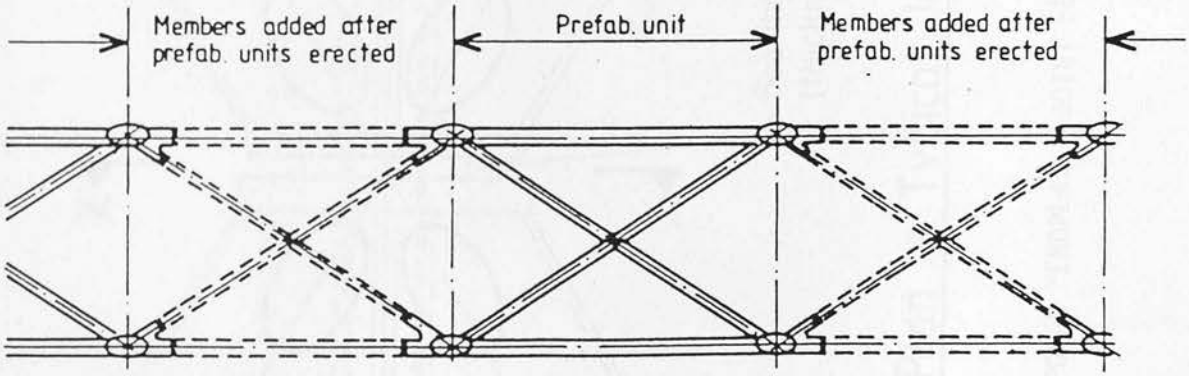


FIG. 16 ISOMETRIC OF TRUSS GRID NEAR A CORNER (BRACING TO "SPIDER" UNITS OMITTED FOR CLARITY)



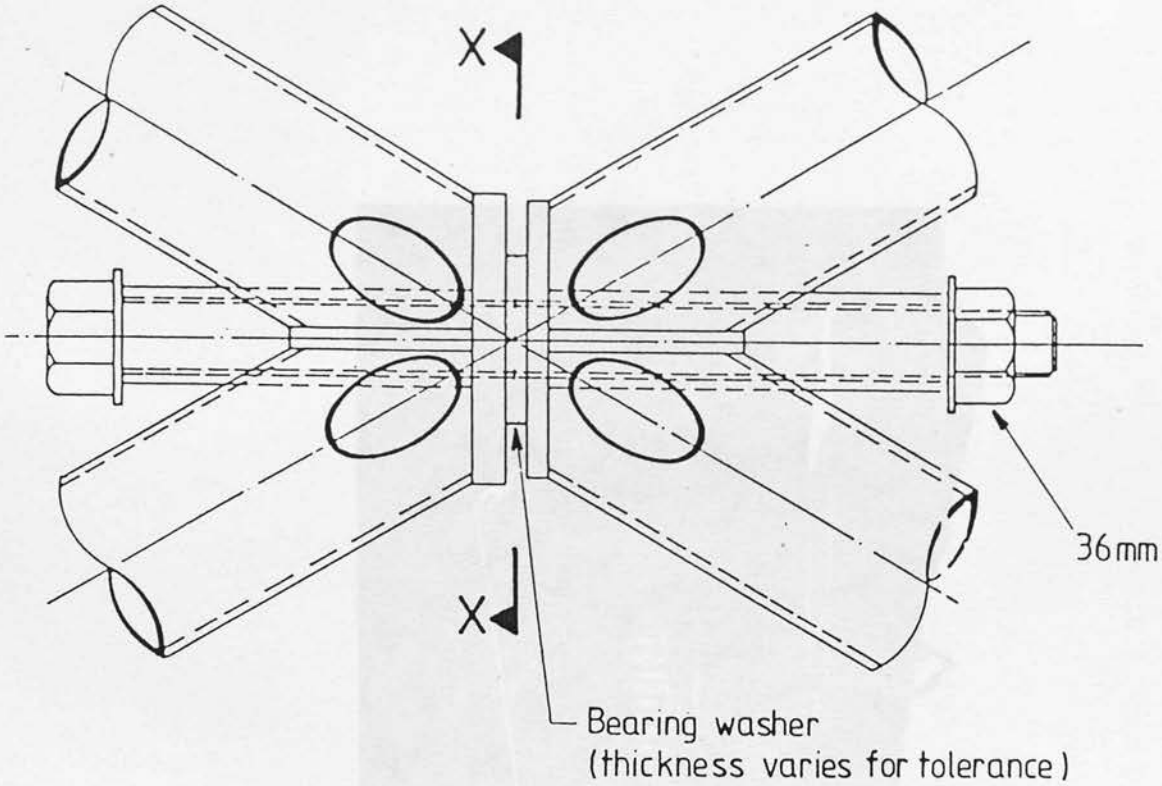


Section 1-1

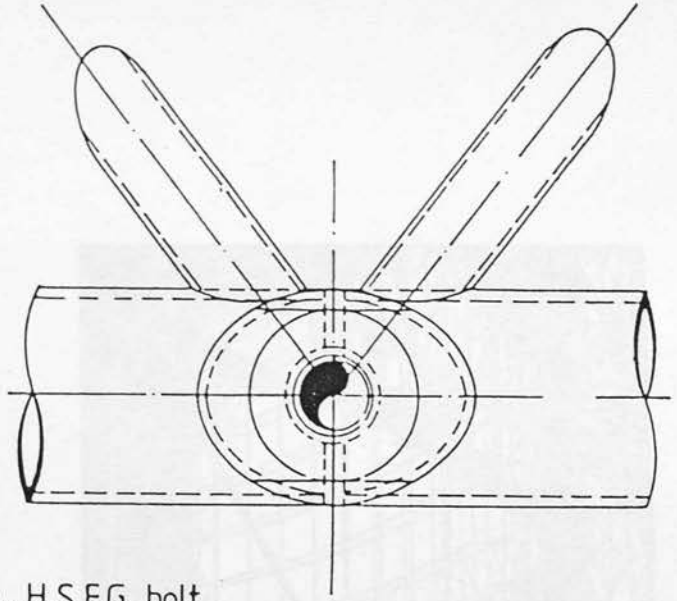


Section 2-2

FIG. 17 PROPOSED "DIAMOND" PREFABRICATED UNIT

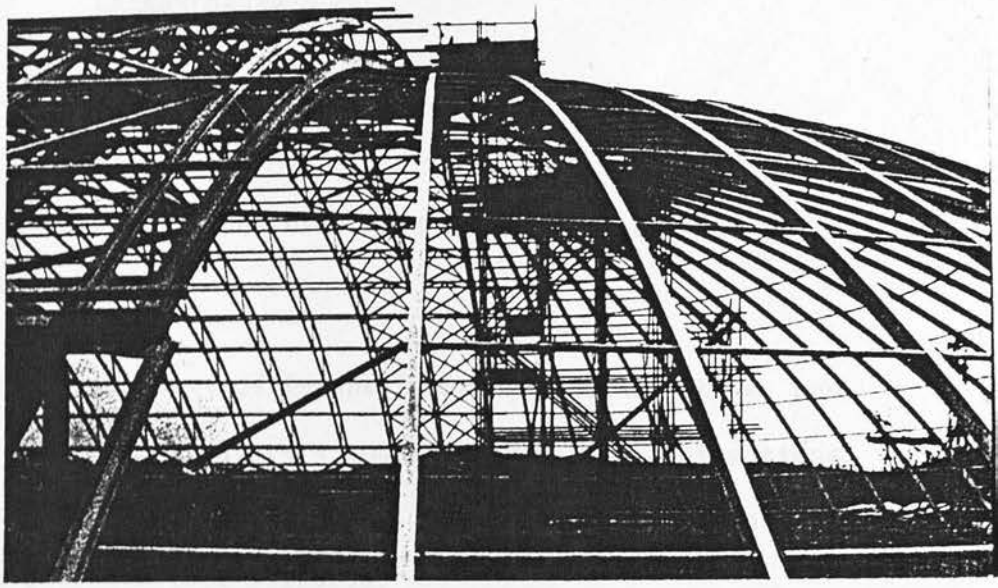


Plan on Typical Joint



Section X-X

FIG. 18 PROPOSED "ENDPLATE" JOINT BETWEEN UNITS



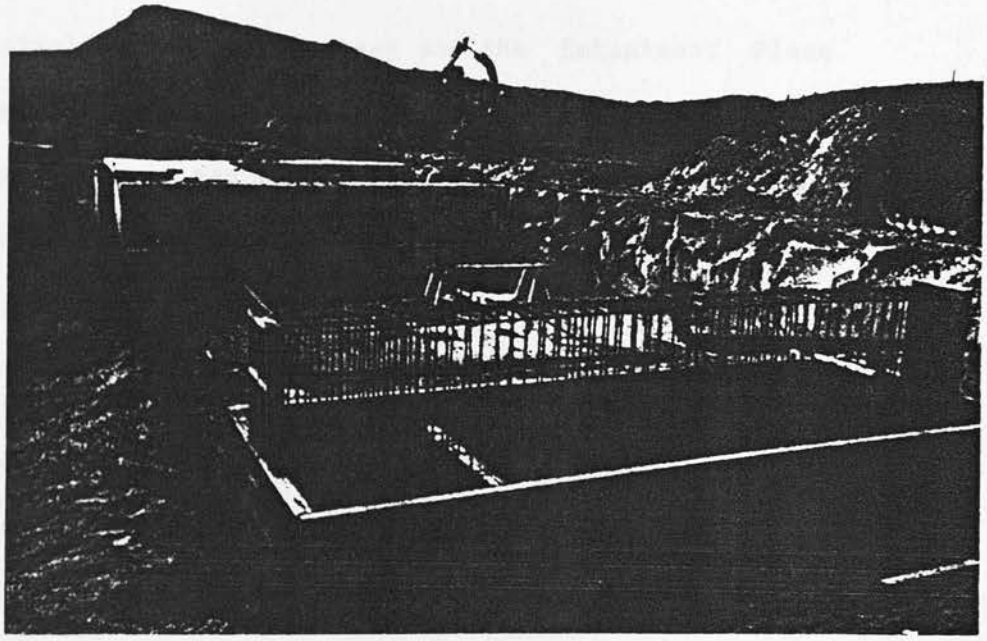


FIG 19

CASE STUDIES 3 AND 4: THE TRANSFER STRUCTURES OF THE AIR RIGHTS BUILDINGS IN THE EMBANKMENT PLACE AND LEE HOUSE DEVELOPMENTS

Introduction

1.0 The use of transfer structures with frame systems is often associated with:

a) Buildings that serve different functions stacked above one another and which require a contrasting range of dimensions and structural spans between floors thus affecting the continuity of vertical supports.

b) Buildings which are designed in tower and podium block compositions, or which are constrained by site requirements and the positioning and number of vertical supports at ground level for example the Lee House development spans over a six-lane dual carriageway and the Embankment Place development involves building over existing railway platforms, and it was required that the construction work was to cause minimum interference, as far as possible to the traffic on both sites. This requirement, as described later, influenced the design development of the structure.

c) The loss of an entire floor occupied by transfer beams of considerable bulk and depth. These floors are often conceived as service floors and have less retail value to the owner.

1.1 The transfer structures of the buildings mentioned in this case study not only resolve the problems described in (a) to (c) but are significant in the following aspects:

i) The architectural form and the desired visual effects were enhanced by the structure.

ii) The transfer structures did not compromise the aim of maximising useable floor area in both commercial schemes.

Site Context

2.0 Lee House

The 18-storey building is astride the London/Wall Street intersection mid-way between Aldergate Street and Moorgate on the City/Barbican Centre boundary. Farrell's design was inspired in part by the medieval Cripplegate which until 1760 stood on the site of the recent Roman House in Wood Street (Figure 1A). The main facade of the building incorporates the appearance of two flanking towers linked together from the first floor upwards by a cylindrical projection and extensive areas of glass surmounted by a shallow curved roof. Figures 1B, 1C show similar themes. The development enhances rather than ignores the existing site: one block bridges the dual carriageway and existing concrete walkways are replaced by half enclosed bridges that are framed by the transfer structure. The transfer structure is unusually located in the same area as those of retail and circulation where a greater degree of visual linkage is required and this implies that the element of the transfer structure cannot be overly bulky or arranged in ways which could cause both physical and visual obstruction (Figure 2).

2.1 Embankment Place Development

This is a major urban scheme aimed at regenerating the Charing Cross area. Sensitively-scaled proposals were preferred to large, out of scale elements in a major site. The scheme comprises a new Air Rights building constructed over the railway platforms of Charing Cross Station, the pedestrianisation of Villiers Street, an extension of the Hungerford footbridge linking Charing Cross with the South Bank, the introduction of arcades on the west side of Villiers Street and the opening up of the existing Embankment Gardens (Figure 3A).

The Air Rights Building itself comprises three principal elements: the main building above the railway platforms, a Villiers Street Block situated between the Charing Cross Hotel and Carrara House, and a canopy structure at the south side of the building (Figure 3B). There are seven floors of deep plan office building on the Thameside frontage rising to nine floors towards the Charing Cross Hotel at the rear. The floor plan dimensions approximate between 50 x 105 m to 50 x 80 m. The building is conceived in the scale of a Thameside palace such as the Palace of Westminster, the National Liberal Club and Somerset House. The building's mass and form is an adaptation of the traditional railway station barrel vault roof, except that the roof is seven to nine floors above ground and the vertical cores to the building are expressed as buttresses to the roof vault. In actual fact, the horizontal thrusts of the roof vault are resisted by a system of prestressed ties so that the main columns receive mainly compression loads from the roof, and there is no need for buttresses. The cores, however provide lateral

stability to the system of floor plates suspended from the roof level arch structure.

Utilitarian Demands and their influence on Architectural Concept

3.0 Lee House

Conventional slab buildings did not meet current demands on quality and size of spaces for the highly serviced needs of computer and electronically operated offices such as Financial Trading Centres. this was because the electronic communications networks, power and air conditioning requirements of computer systems require a system of false floors for flexibility in routing and layout - a workstation in the centre of a deep floor plan office cannot be satisfactorily connected to a wall mounted power point, and suspended network cables interfere with movement and are not aesthetically satisfactory. The number of penetrations in floor slabs required for electrical and network ducting purposes would render the slab structurally ineffective. Therefore a system of lightweight, fire-proof and finished floor panels raised on metal stumps bolted to the structural floor slab is required and this effectively increases the floor to floor height which would then reduce the number of floors that can be provided within a given height constraint for a building. In the case of commercial buildings, it would imply less retail space and this problem is further aggravated by deeper floor beams for schemes with a minimum of vertical support. Therefore, the structural scheme had to respond to these conflicting requirements in order to make the proposal a feasible one.

3.1 Embankment Place Development

This development is proposed on a site with high commercial value due to its Thameside frontage and the incorporation of Charing Cross Railway Station, combining both transport and commercial functions in one building. The proposal of building useage includes shops, offices and a financial trading floor which requires large floor areas free from structural encumbrance.

4.0 Constraints

The structural design of the Air Rights buildings to be constructed over two very busy parts of London City had to respond to a number of constraints. The scheme design structural solution therefore evolved from a series of studies which addressed these constraints in connection with those of the structure. The resulting structure not only takes into account these parameters but also sets out to satisfy the planning expectations of a large prestigious city office building.

4.1 Embankment Place Development

One of the main aims of the design has been to satisfy the requirements of British Rail who not only operate the railway terminus but are also the head landlords for the occupiers of the vaults beneath. BR's prime concerns are for the safety and comfort of their passengers and for the retention of the structural integrity of the station at all times.

Charing Cross station has three times as many passengers per platform than any other London Station. British Rail therefore requires that minimum disruption be caused to the platforms both

during and after construction. Although this constraint is not quantifiable, it determines BRR's priorities for this development. The existing brickwork vault structure which supports the station platforms and tracks is highly stressed and sensitive to any changes in load paths.

Alterations to the vaults have been minimised and have been restricted to areas which will least affect the existing structural action.

In addition to these constructions, the location of main support columns to the building and their foundations are governed by a number of physical barriers:

a) The minimum clearance from the platform edge must be at least 2 m with a preferred 3 m stipulation by British Rail.

b) The location of the Northern Line Underground tunnels which pass under the North East corner of the site (Figure 4).

c) The maintenance of a twenty feet statutory Right of Way at all times within Craven Passage which passes east-west through Vault no 179 under the station (Figure 4).

d) The avoidance of the existing structures that have been underpinned as a result of mvoements caused by past changes in the ground water level.

e) The maintenance of complete structural separation between all new construction and existing structures.

f) The provision of a minimum amount of new vertical structure in the following areas within the vaults: The Players Theatre, the service bay and service road, and the entrance foyer.

g) The provision of new retail units in the vault areas.

h) The height of the building should not exceed the existing roof line of the Charing Cross Hotel.

i) The level of the lowest office floor is governed by the minimum clearance to the station platform level as set out by British Rail.

British Rail also required that no work be carried out above platforms or tracks in use and this influenced the method of construction up to and including the level 5 podium slab. The railway tracks were consequently closed on weekends to permit construction and a structural floor slab over the platforms as protection during construction work.

4.1.1 Development of Office Structure (Options)

During the feasibility study, various options were considered for the transverse structure of the Air Rights Building, and they fell into three main categories:

a) Six column structural frame (Figure 5A).

b) Four column structural frame (Figure 5B).

c) Two column structural frame (Figure 5C, D).

All the solutions assumed the same building envelope and all have responded in varying degrees to the constraints imposed by the site and the design brief. The longitudinal spacing of the columns is governed by constraint (c), the vault layout, and the underpinned foundations at the southern end of the vaults. These constraints were common to the three categories of transverse structural options and their study resulted in the use of a common North-South column grid spacing of 12 m, 10.5 m, and 9 m.

4.1.2 Six Column Frame

The six column frame represents the maximum number of columns that may be possibly positioned on the platforms, in compliance with British Rail's platform clearance requirements (Figure 5A). The advantage of this option is that it is a conventional long span solution without the need for transfer structures. The disadvantages are that the irregular spans make office planning and the location of atria difficult. (The irregular spans were a result of locating columns outside station walls and vault structures, and meeting the platform clearances). The long spans require large structural depths at each floor which could result in the loss of one floor due to the height restrictions in constraint (h). British Rail were not in favour of the large number of columns which could cause considerable disruption to the operation of the platforms, and would require extensive alterations to the existing vault

structure. Disruption was also anticipated in Craven Passage, the service road, the Players Theatre and the entrance lobby. Where the proposal did not require transfer structures to support the main office floors, they were required at platform level to prevent columns running through the tube tunnels and the existing underpinning, and they would also be required for the external row of columns adjacent to Carrara House.

4.1.3 Four Column Frame

The four column frame is a refinement of the six column frame, which solves some of the difficult problems associated with it. Two external columns have been deleted with the inclusion of a transfer structure one level above the railway platforms, providing a regular 6 m grid appropriate to office planning (Figure 5B). The advantage is that the solution achieves a regular and orderly planning grid for the office structure, which the six column solution could not.

The disadvantages were that the number of columns passing through the platforms were still unacceptable to British Rail and there would be extensive alterations to the existing vault structures and considerable disruption caused to the Players Theatre and Craven Passage. Aso, a second transfer structure required at Level 6 to create a column spacing of 12 m for the proposed financial trading floor at level 5 would result in the loss of one office floor.

4.1.4 Two Column Frame

The two column frame accommodates the majority of the constraints imposed by the operation of the railway terminus, the structure and the future use of the vaults. The main vertical supports are reduced to two rows of columns on platforms at the far ends of the terminus, thereby minimising obstruction to the operation of the platforms and minimising disruption to the existing vault structure beneath (Figure 5C).

To do this, the office superstructure had to become a more complex structural solution, and consequently, costlier than the other alternatives. The design engineers claim that this extra cost was offset by simpler foundations.

Two families of structural solution were studied. The first utilised the framing structure as a whole to span the 36 m between the two primary columns, and the second isolated the transfer structure as a discreet element. Within the first family, two options were considered:

a) A triangulated framework with the apex at roof level.

b) Consideration of the entire building as a vierendeel frame.

The triangulated framework had the advantage of efficiently sized members but involved the design and construction of complex joints which had to be formed in-situ. The sloping columns interfered with useable space and the subsequent loss in retail value was unacceptable to the developers.

The vierendeel framework which utilised a conventional orthogonal arrangement of beams and columns was structurally inefficient because joint sizes had to be disproportionately large in order to obtain the required overall frame stiffness. Within the second family, again two options were considered:

c) A low level transfer structure at podium level comprising a floor depth triangulated truss.

d) A high level transfer structure consisting of a tied arch at roof level from which the office structure is suspended.

The study of option (c) reached similar conclusions as option (a) with the unacceptable loss of retail space and the encumbrance of diagonal members in the financial trading floor.

The roof level tied arch was considered the best solution in terms of structural efficiency and appropriateness to the architectural requirements of the building. The floors were suspended from the tied arch and there was no need for a low level transfer

structure, which subsequently freed the trading floor from structural encumbrance. The solution also allowed the floor framing to be standard.

4.1.5 Office Structure

The main columns consist of 1 m diameter rc cast within 15 mm thick steel permanent formwork, from foundations up to level 5 and above level 5, they comprise heavy steel sections, connected to the rc columns by means of a large base plate and holding down bolts. Below level 5, the columns have been positioned to allow penetrations through the existing vault structure, close to the crown of the arch whilst maintaining a logical structural grid. The penetrations are designed to separate new columns from the existing structure. At roof level, the main columns support the tied arch transfer structure spanning 36 m from which all floors are suspended.

The hangers comprise steel universal column sections with hinge connections to the arch. The edge cantilevers at roof level will be formed by a triangulated system of steel members which will utilise an extension of the arch tie as the tension member.

The floor plates are supported by the main columns and hangers and they will consist of a system of primary and secondary steel beams acting compositely with a leighweight concrete floor slab. The floor plates also

act as a horizontal diaphragm to transfer the horizontal loads to the cores which provide the lateral stability.

4.1.6 Arch Structure

The design engineers were of the opinion that the circular arch was not efficient in terms of accommodating out-of-balance loading, and the ease of fabrication and erection. In response to the distribution of forces within the arch, its steel section is made deeper at the crown than at the supports. Vertical support to the arch at the main columns is provided by means of bridge bearings which permit rotation and horizontal movement at one end and rotation only at the other. The bearings ensure that the horizontal thursts of the arch are not transmitted to the rest of the framework.

To control the forces and deflections of the arch, the springing points are connected by a system of prestressed high strength steel bars. The anchor block for the ties is located above the bearings. Because a major component of the vertical deflection of the arch is that attributed to the elongation of the tie, the tie itself was prestressed in stages as the load to the arch increased.

(This aspect of its construction will be discussed in the following paragraph on structural action). The arch carries the hangers which support the office floors from level 5 to 13. The lateral stability of the arch is provided by trusses fabricated from hollow steel sections spanning between the arches together with one braced bay at roof level which will transfer the lateral loads to the cores.

4.1.7 Structural Action

The tied arch transfer structure will deflect under load and this deflection has two major components: those due to the weight of the structure and those due to the imposed loads applied to it. The structural design has concentrated on the elimination of the first component and the control of the second.

During construction the frame structure will be bottom supported by both temporary and main columns. In order to transfer the loads from the temporary columns to the arch, the arch tie will be prestressed. This will cause the arch to deflect upwards to take up the extensions in the columns/hangars as their stresses change from compression to tension. As the floor slabs are concreted in sequence, the arch ties will again be prestressed in stages until the full weight of the structure is transferred to the arches and the temporary columns removed. In this way the slabs will not change in level as the dead loads are transferred

to the arch.

The second component of the arch deflection caused by imposed loads was not controlled by prestressing and the arch has been sized to keep this component at a level that is within normal building tolerances.

4.2 Lee House Design Constraints

The major structural task was to design a transfer structure to span a six-lane dual carriageway of London Wall. This was complicated by the fact that at ground level, two of the columns would have to be located on the islands in the middle of Wood Street, one of which was set back from the edge of the adjacent pavements running along London Wall (Figure 7, 8, 9). This meant that the necessary minimum distance between columns at ground level varied between 21 m and 30 m. Being a major road junction, it could only be closed on weekends and the construction and design of the structure had to take this constraint into consideration. The floor levels of the proposed building had to match those of the existing building, as the scheme involved physically linking the two together.

Development of Transfer Structure (Lee House)

Because no columns were permitted in the central areas of the ground floor plan in order to permit the uninterrupted flow of existing traffic, one solution would have been to put columns on either side of the building and have every floor spanning the road, but that would have implied deep beams at every floor which would have reduced the number of floors possible within the fixed height of the building.

The use of deep beams at every floor would also have made it difficult to match the floors of the proposed building with those of the existing one. There were two alternatives to transferring the weight from the proposed four columns structure of the upper levels to a two-column structure on either side of the road level: one was to suspend the structure from a beam at roof level and the other to support the structure on a beam over the road. This meant that a transfer structure was needed at either the top level of the building or at the first floor level of the building. In this case, practical considerations led to the location of the transfer structure at first floor level. This was because the erection sequence for the 18-storey Lee House Air Rights building started from 'bottom-up' ie the services and cladding would be installed from the base of the building upwards as the structure to the upper storeys was being constructed. This implied that a podium level transfer structure was necessary to accommodate this sequence and obviated the need to build a temporary platform over the road during construction.

There were also architectural reasons that led to the positioning of the transfer structure (Figure 10). The building form involved the stepping back of the roof line for reasons of architectural massing and building envelope restrictions prescribed by planning guidelines. A roof-level transfer structure would have been partially exposed by the stepping back of the roof line, and was not appropriate to the intended expression of the architectural form (Figure 6).

The transfer structure was to occupy the retail level of the proposed building with pedestrian galleries and circulation area which required a great degree of visibility between spaces in order to function effectively. There was thus an unusual constraint on the transfer

structure to be as minimal as possible in order to integrate well with its adjacent retail functions of space. The transfer structure at the end elevations of the podium was to provide the visual cue of an archway in the podium facade, this expression drawn from the Cripplegate (Figure 1A to 1C).

Figure 11 to 18 illustrates the options considered in the development of the transfer structure. The requirement that the first floor public area should be as far as possible free from columns, precluded the use of a three-dimensional space frame so various forms of trussing were considered. Along with the other standard designs the traditional trapezoidal truss in which the upper chord was shorter than the lower chord was discounted because the diagonal lateral struts would have obstructed movement along the walkways at the sides of the pedestrian area. To get around this problem, a truss where the traditional trapezoidal form was inverted was used. The final system adopted was one with cable-stayed trusses where adequate headroom was provided within the structural configuration, and two additional bowstring arches at either end where the entrance to the tunnels were to be marked by archways. The high arches were intended to reduce the visual impact of entering the tunnel and the trusses were appropriate to the architectural aim of creating a suspended box over the road.

Development of the Lee House Podium Structure

The following is an excerpt from a report prepared by Neil Noble, OAP.

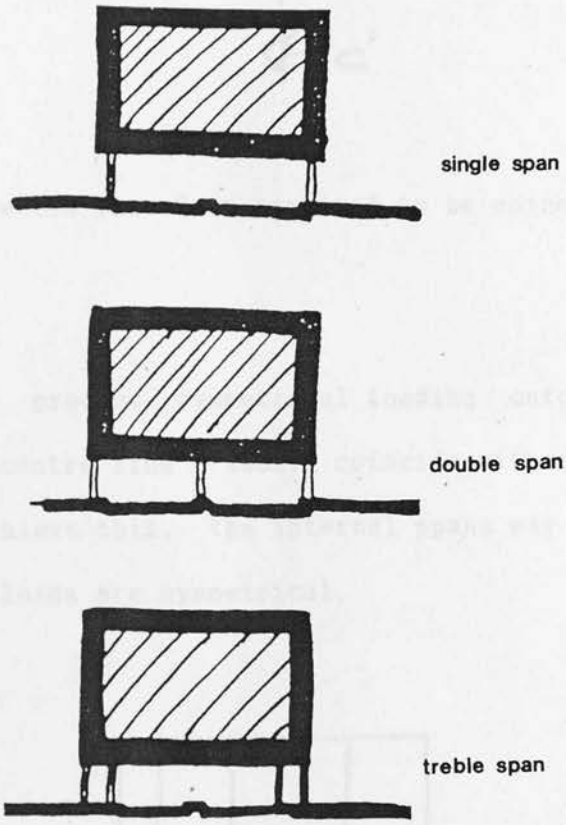
FIGURE 7

Development of the London Wall Podium Structure

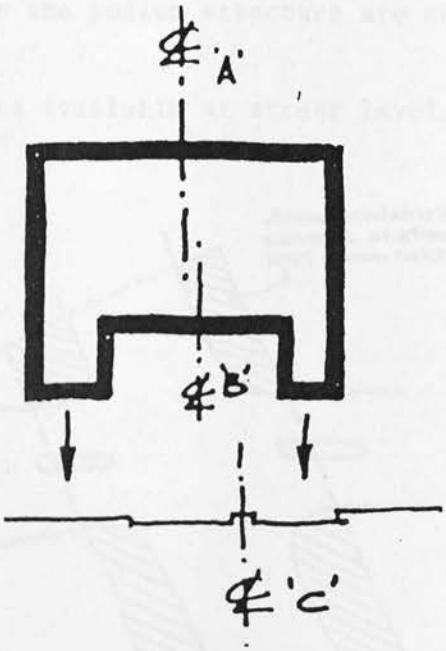
The Objectives

The Podium structure is required to:

a) Span London Wall with either 1, 2 or 3 spans, and to support the superstructure of the Air Rights Building above.



b) Be geometrically compatible with the road below for visual reasons, and the building above for symmetry of loading.



Visually, centre line B is required to be coincident with centre line C.

Ideally, to produce symmetrical loading onto the supporting structure, centre line A should coincide with centre line B. In order to achieve this, the internal spans may be adjusted, so that column loads are symmetrical.

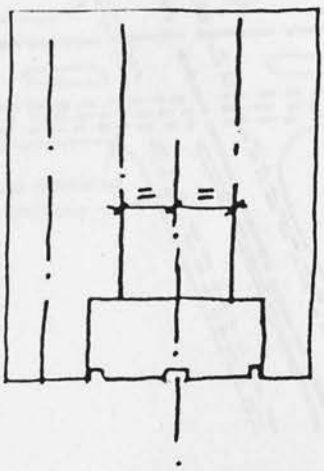
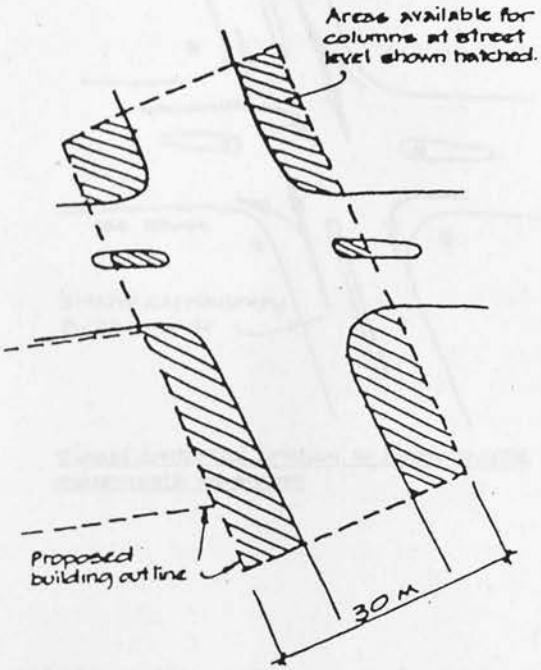


FIGURE 8

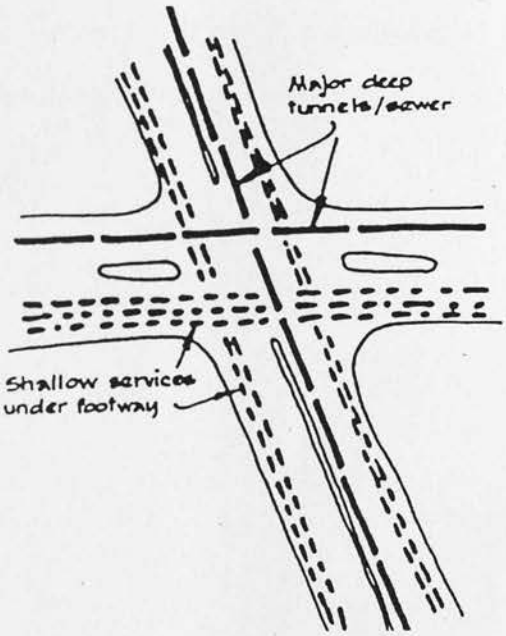
The Constraints

Column positions for the podium structure are constrained due to:

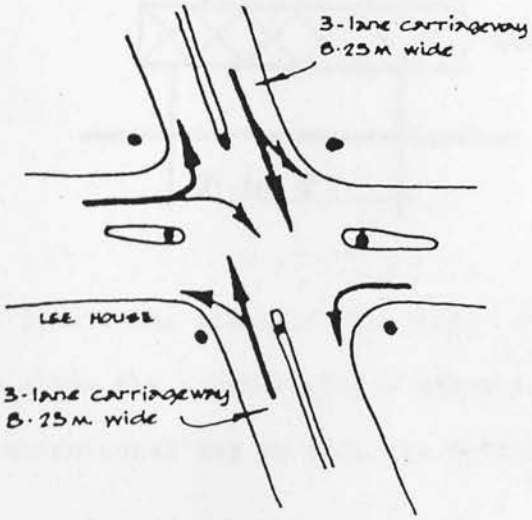
a) The limited area available at street level.



b) The need to minimise services and tunnel disruption/diversion.



c) The required traffic management of the London Wall/Wood Street intersection, to be adjusted to take into account projected traffic movement.



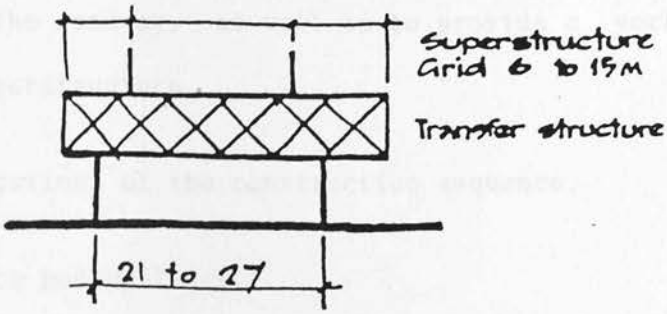
Signal controlled junction to allow traffic movements as shown

d) The need to maintain the entrance and exit ramps and the capacity of the existing car park below London Wall.

FIGURE 10

Development of the Transfer Structure

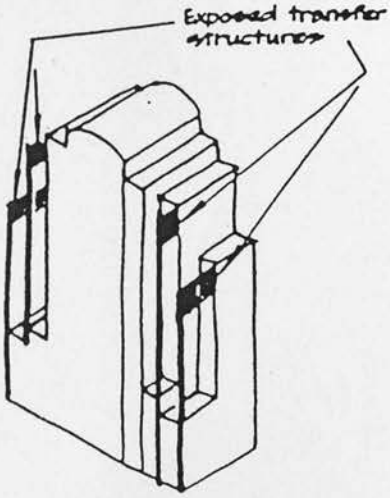
Location



It was decided to locate the transfer structure at around podium level, in order to allow the superstructure above this level to be constructed in the conventional way ie from the bottom up.

The alternative solution of taking the main load bearing columns up to roof level and suspending the superstructure from a transfer structure at this level was abandoned due to:

a) The profile of the building does not allow a regular arrangement of high level transfer structures spanning London Wall, since the roof level varies along the length of the building. Any high level transfer structure would therefore be exposed in certain areas. This was considered unacceptable, architecturally.



b) Impact on floor plans of the large primary load bearing columns, plus the smaller hanging columns.

c) The need for a temporary transfer deck above road level to allow continued use of the roadway, as well as to provide a working platform for the superstructure.

d) The programme implications of the construction sequence.

- Erect columns to podium level.
- Construct temporary deck at podium level.
- Erect superstructure off temporary deck to roof level.
- Transfer load from temporary deck to roof transfer structure.

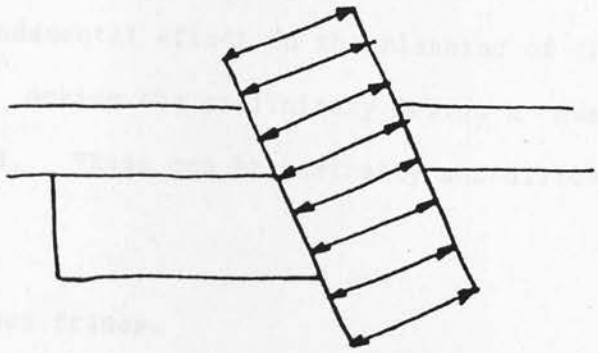
e) Building movements associated with this sequence on following trades eg cladding, would need to be allowed for in the design and very carefully monitored during construction.

FIGURE 11

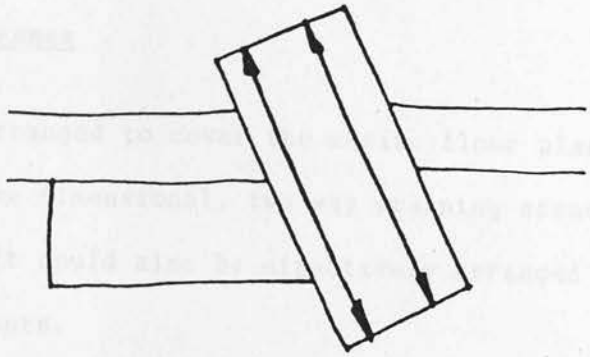
Arrangement

The various arrangements of the primary structural elements were examined. These were:

a) Transverse



b) Longitudinal



c) Diagonal

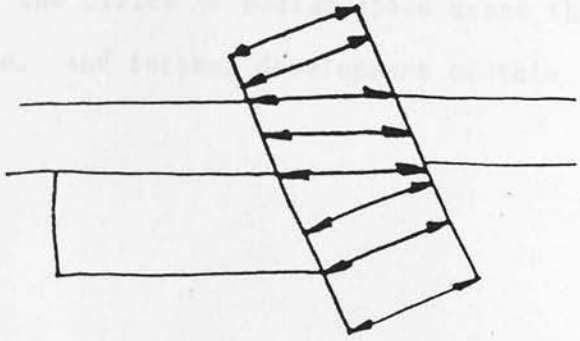


FIGURE 12A

Form

In view of the very large loads carried by this transfer structure, it needs to be one or more floors deep. For this reason, the structural solutions will have a fundamental effect on the planning of the floor on which it is located. during the preliminary design a number of options were investigated. These can be basically sub-divided into two categories:

a) Three dimensional space frames.

b) Two dimensional plane frames.

Three Dimensional Space Frames

The space frame may be arranged to cover the entire floor plan of the building, and being a three dimensional, two way spanning structure is an efficient solution. It could also be effectively arranged as two primary longitudinal elements.

However, the rigidity of the space frame relies on the diagonals between the upper and lower planes. The major impact these elements have on the organisation of the office or podium space means that this is not a practical solution, and further development of this option was therefore abandoned.

Two Dimensional Plane Frames

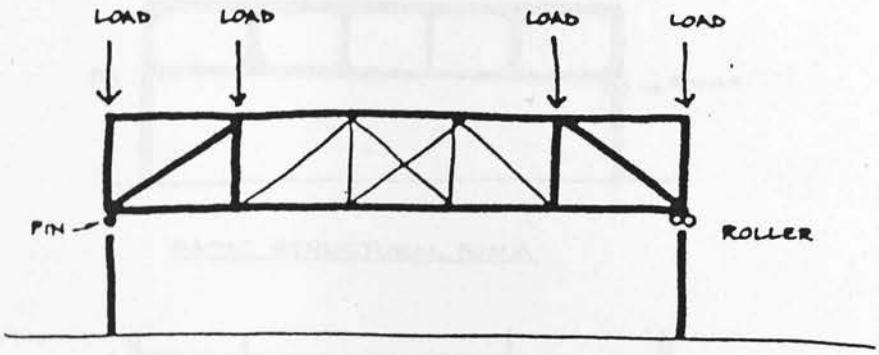
During the preliminary design, a number of two dimensional frames were examined. Basic sketches of these are shown overleaf.

Due to the clearance required for traffic movement, it was decided that the structure between road level and podium soffit could only be vertical, and for this reason, the deep beam, the vierendeels, the bow string and the A-frame options, which can be supported by vertical columns alone, were developed further.

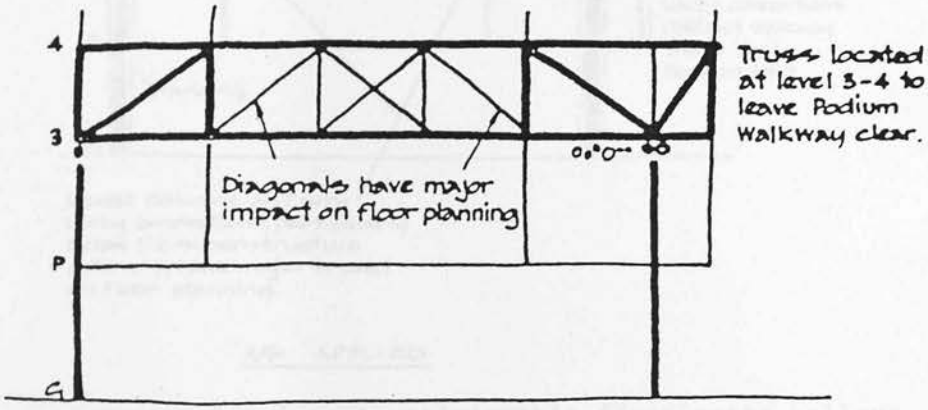
FIGURE 13

Comparison Between The Options

Deep Beam Long Span



BASIC STRUCTURAL FORM



AS APPLIED

Advantages — Structurally efficient.

Relatively simple to fabricate/erect.

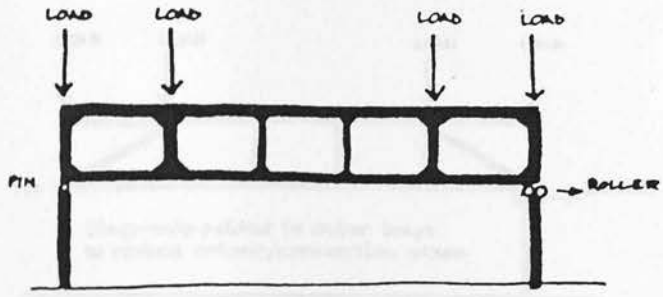
Disadvantages — Requires full storey height to support superstructure loads.

The diagonals have a major impact on space planning of that floor.

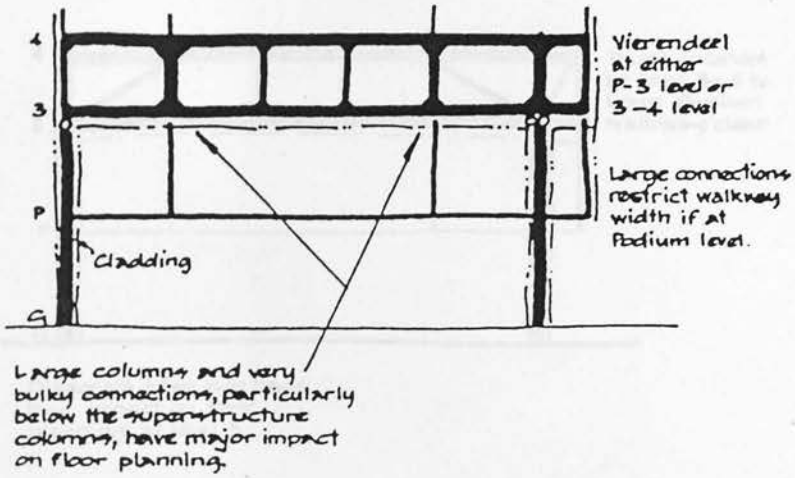
Approx weight — Estimated weight of structure based on preliminary design is 125 tonnes.

FIGURE 14

Vierendeel - Long Span



BASIC STRUCTURAL FORM



AS APPLIED

Advantages

Vertical elements only within floor space allows greater flexibility of planning.

Disadvantages

Structurally inefficient.

Due to the magnitude of the loads and span, the member forces are high - connection details are very complicated and extremely large.

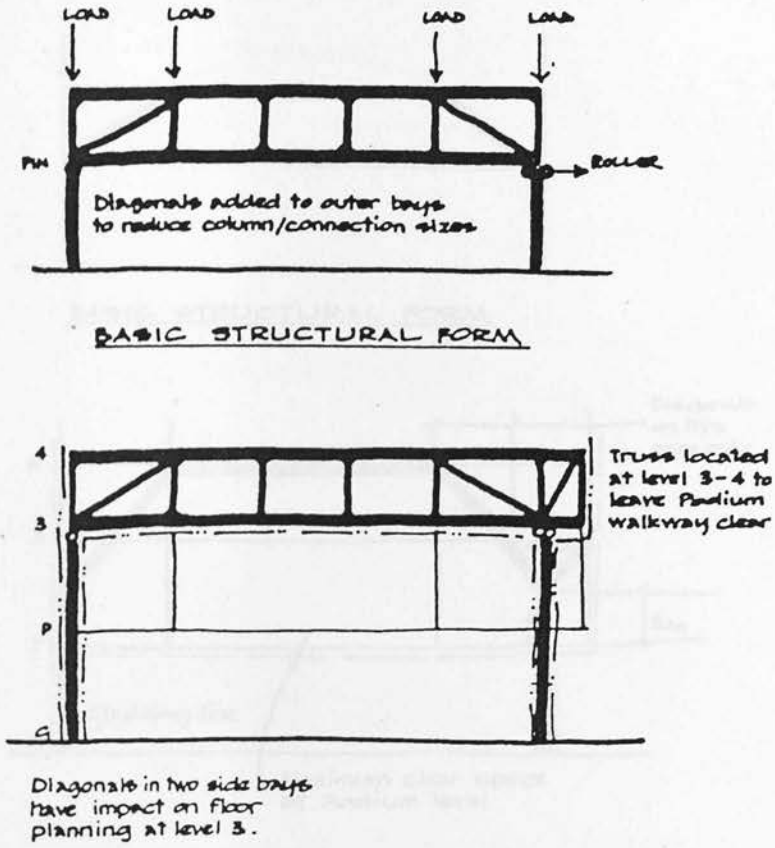
Member sizes, columns within the office space are large.

Approx weight

Estimated weight of structure based on preliminary design is 250 tonnes.

FIGURE 15

Braced Vierendeel - Long Span



Advantages

Retains the basic space planning flexibility of the pure vierendeel in the centre of the structure.

Bracing elements, located in end bays only improve the load carrying characteristics, and thereby reduce connection details.

Disadvantages

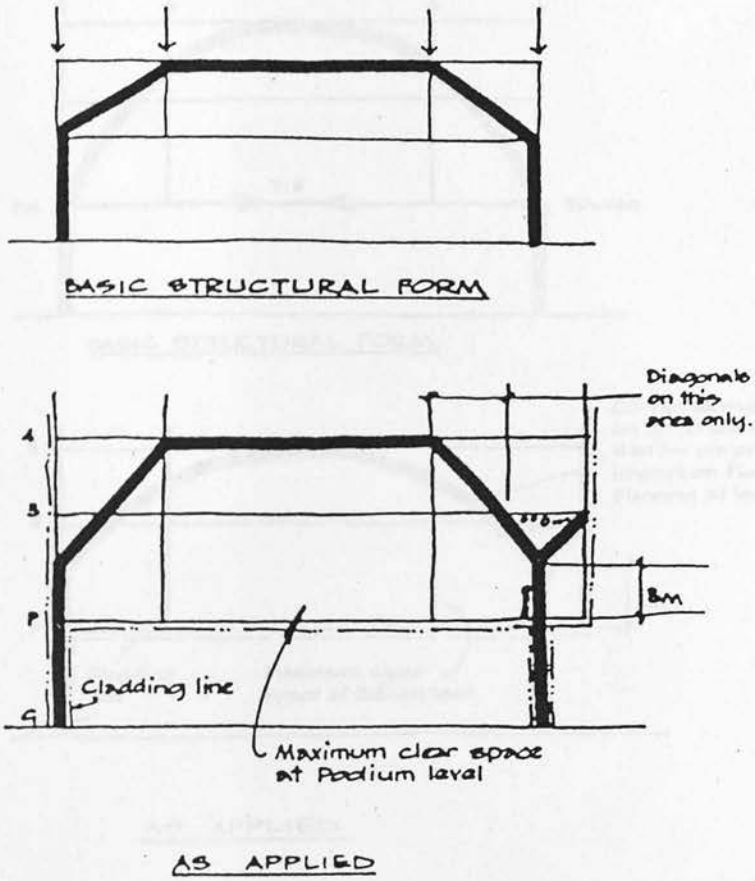
End bays containing verticals and diagonals will have impact on space planning in these areas.

Approx weight

Estimated weight of structure based on preliminary design is 160 tonnes.

FIGURE 16

A-Frame - Long Span



Advantages

Like the braced vierendeel, the diagonal elements are located in the end bays only and therefore the effect on the basic space planning is minimised.

Disadvantages

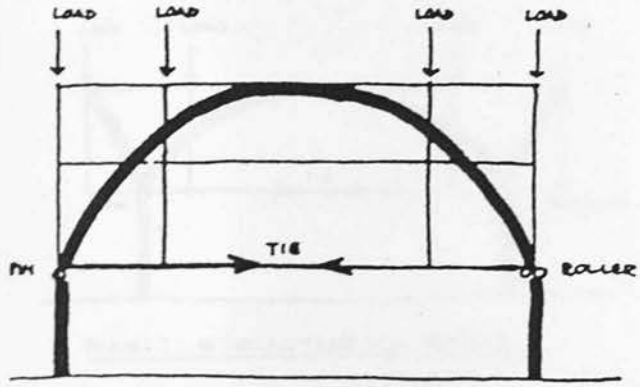
Primary horizontal and diagonal elements are deeper than for the braced vierendeel, and may result in reduced headroom, (possibly only below the frames, depending on the false ceiling details).

Approx weight

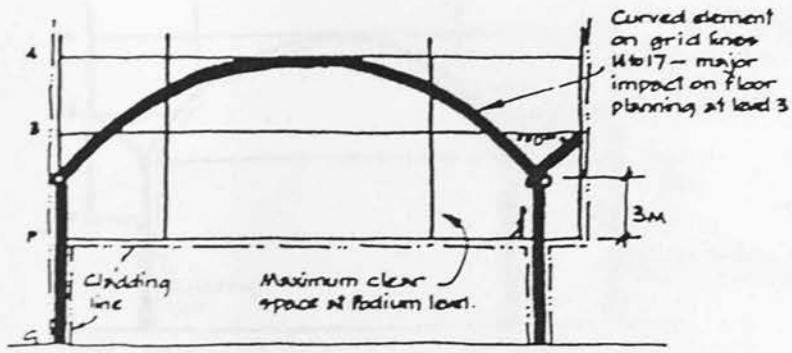
Estimated weight of structure based on preliminary design is 120 tonnes.

FIGURE 17

Bowstring - Long Span



BASIC STRUCTURAL FORM



AS APPLIED

Advantages

The shape of the primary structure accurately and simply reflects the structural performance required, and therefore results in an economical solution.

The curved shape is compatible with the architectural elevations.

Disadvantages

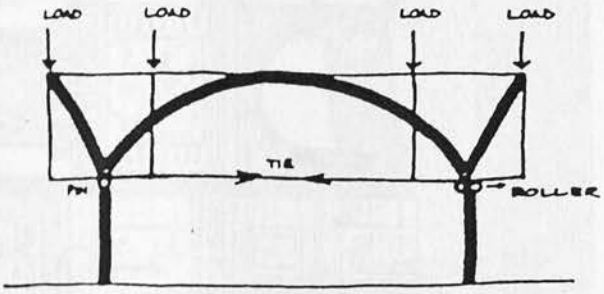
Depending on the rate of curvature, and the floor to floor height, there could be a serious impact on the floor planning.

Approx weight

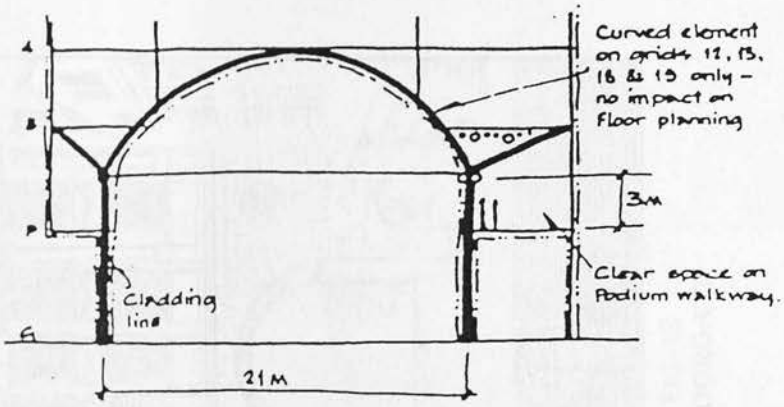
Estimated weight of structure based on preliminary design is 165 tonnes.

FIGURE 18

Bowstring - Short Span



BASIC STRUCTURAL FORM



AS APPLIED

Advantages

The shape of the primary structure accurately and simply reflects the structural performance required, and therefore results in an economical solution.

The curved shape is compatible with the architectural elevations.

Disadvantages

Depending on the rate of curvature, and the floor to floor height, there could be a serious impact on the floor planning.

Approx weight

Estimated weight of structure based on preliminary design is 80 tonnes.

FIG 1A
CRIPPLEGATE.

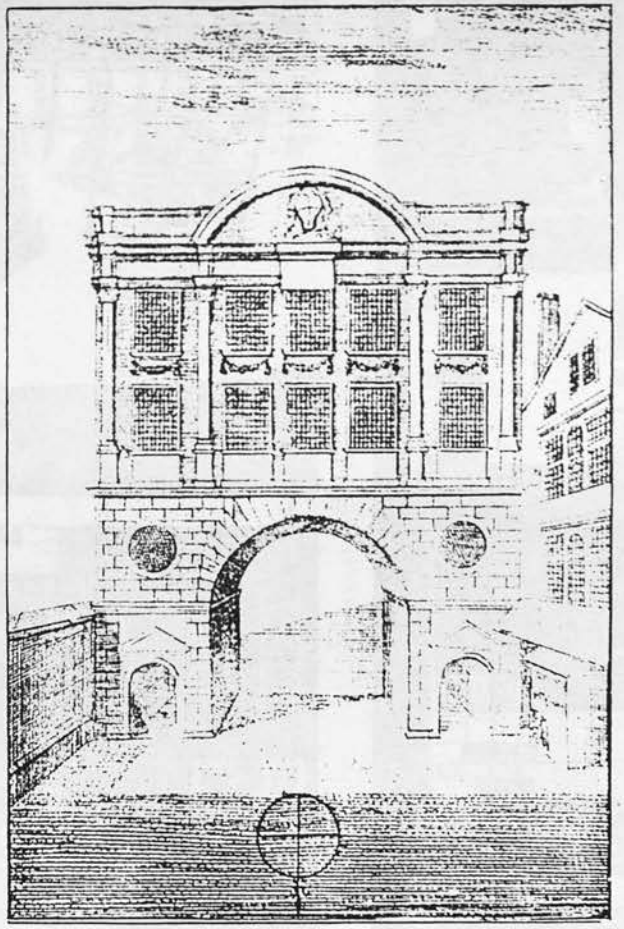
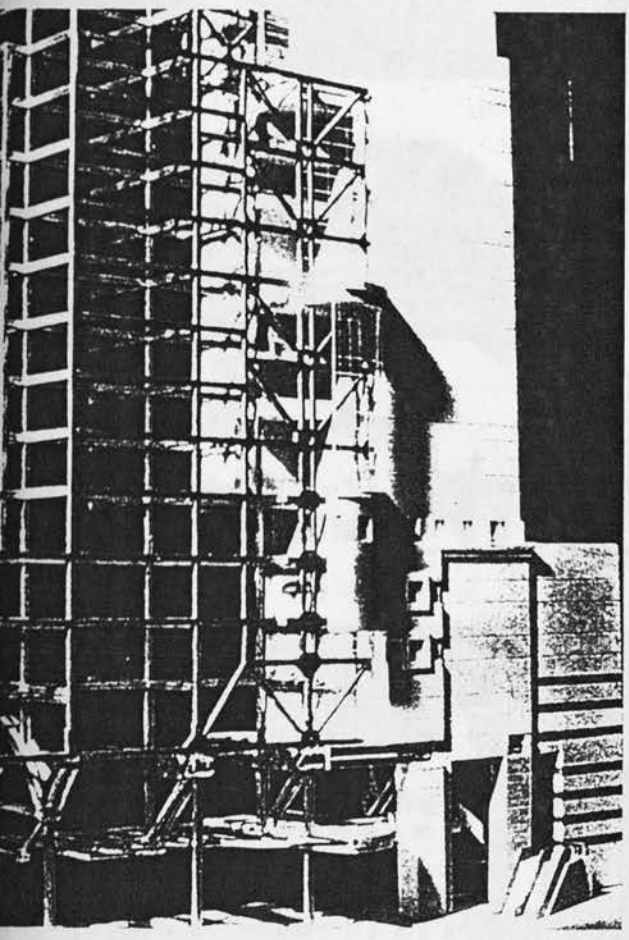


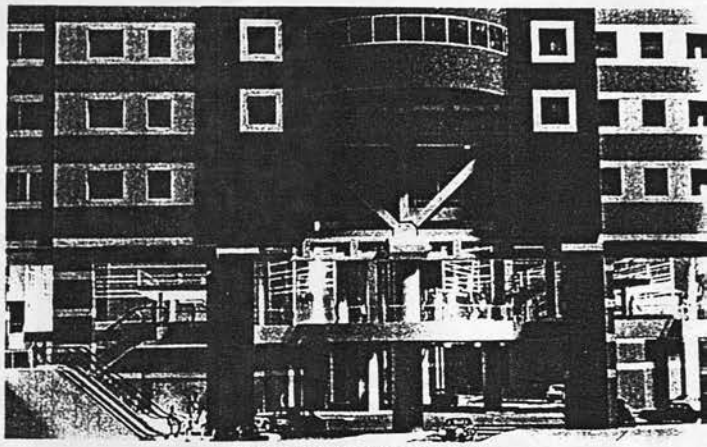
FIG 1B
MOORGATE

FIG 1C
ALDERSGATE : THE OUTER FRONT.

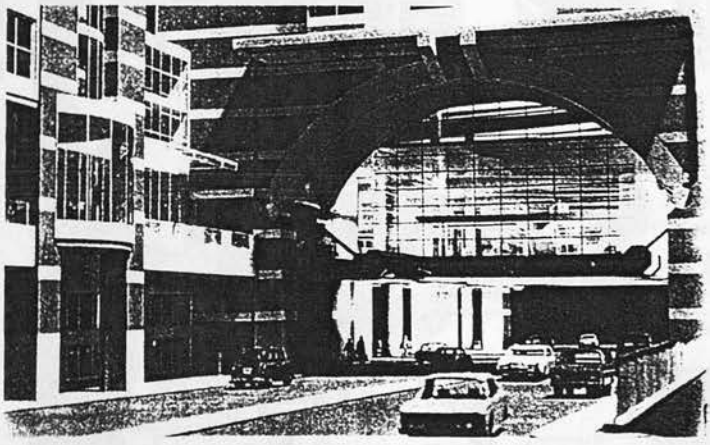
LEE HOUSE SIDE ELEVATION



LEE HOUSE, MODEL OF INTERNAL STRUCTURE

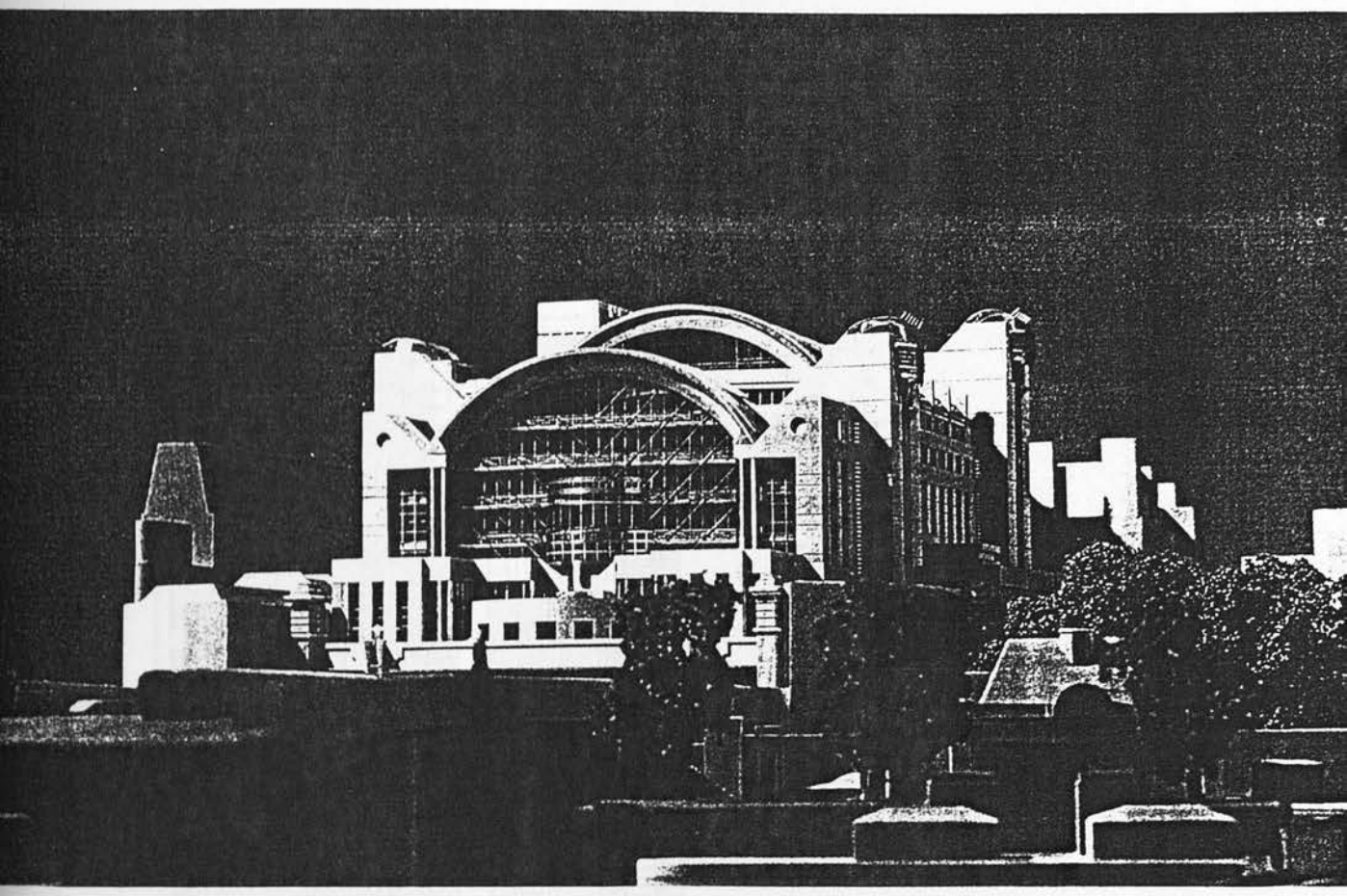


LEE HOUSE, MAIN ENTRANCE

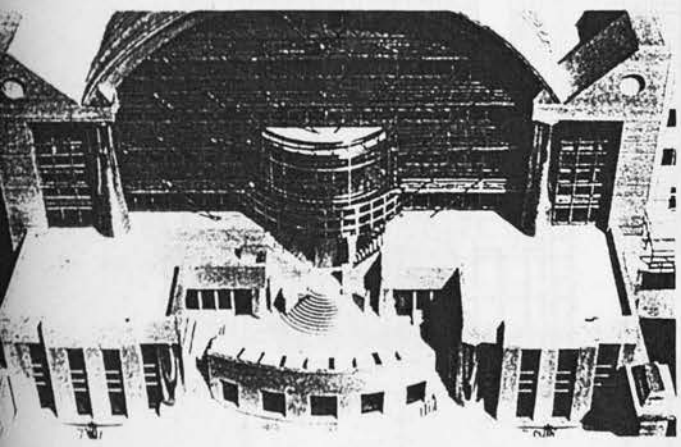


LEE HOUSE, MODEL BRIDGING LONDON WALL

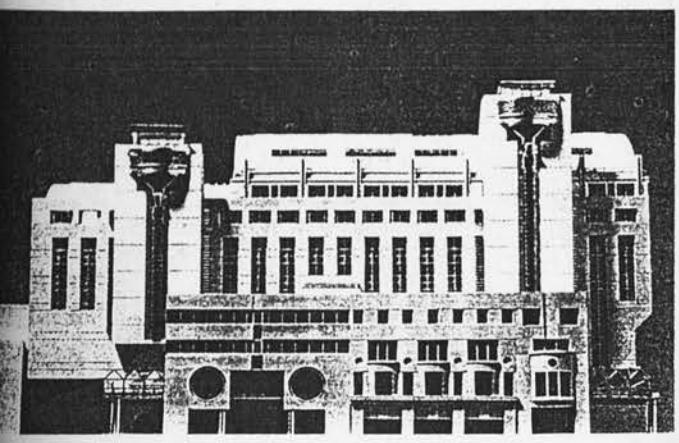
FIG 3A



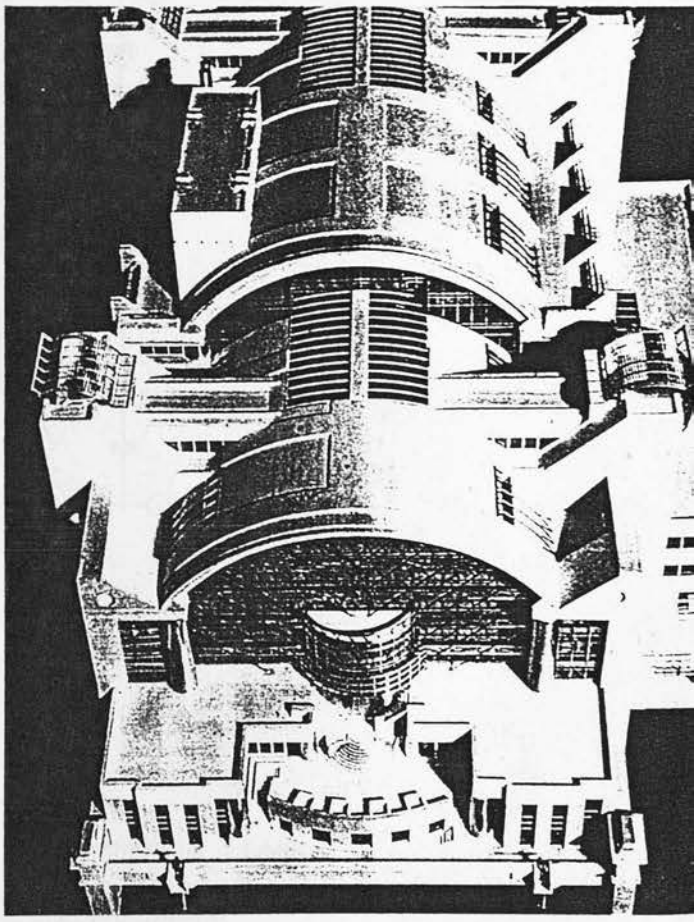
EMBANKMENT PLACE THAMESIDE ELEVATION



THAMESIDE FACADE AIR RIGHTS BUILDING



EMBANKMENT PLACE VILLIERS STREET ELEVATION



AERIAL VIEW OF EMBANKMENT PLACE

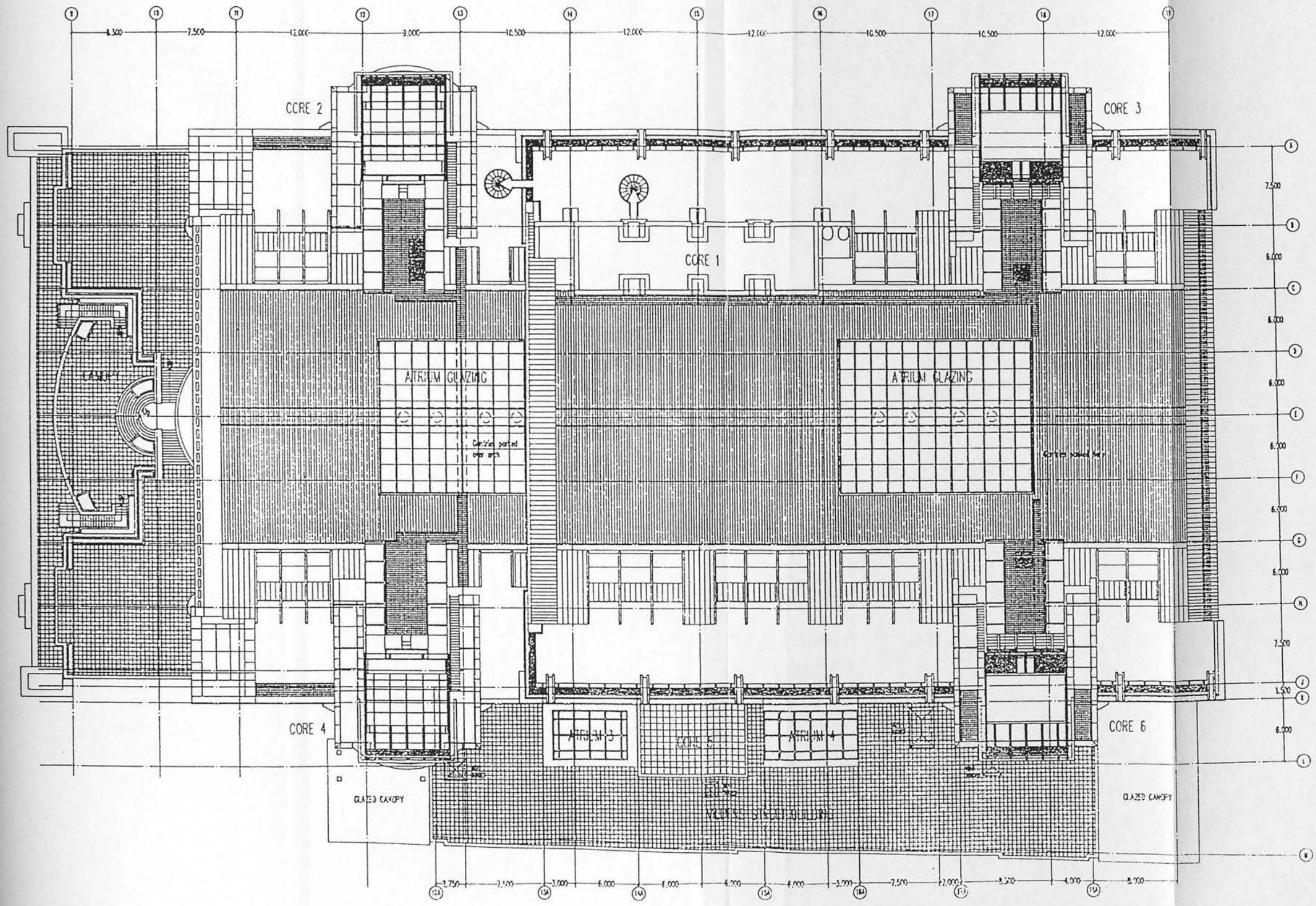


Fig 3B
EMBANKMENT PLACE
ROOF PLAN
1:400 SCALE
20-07-88

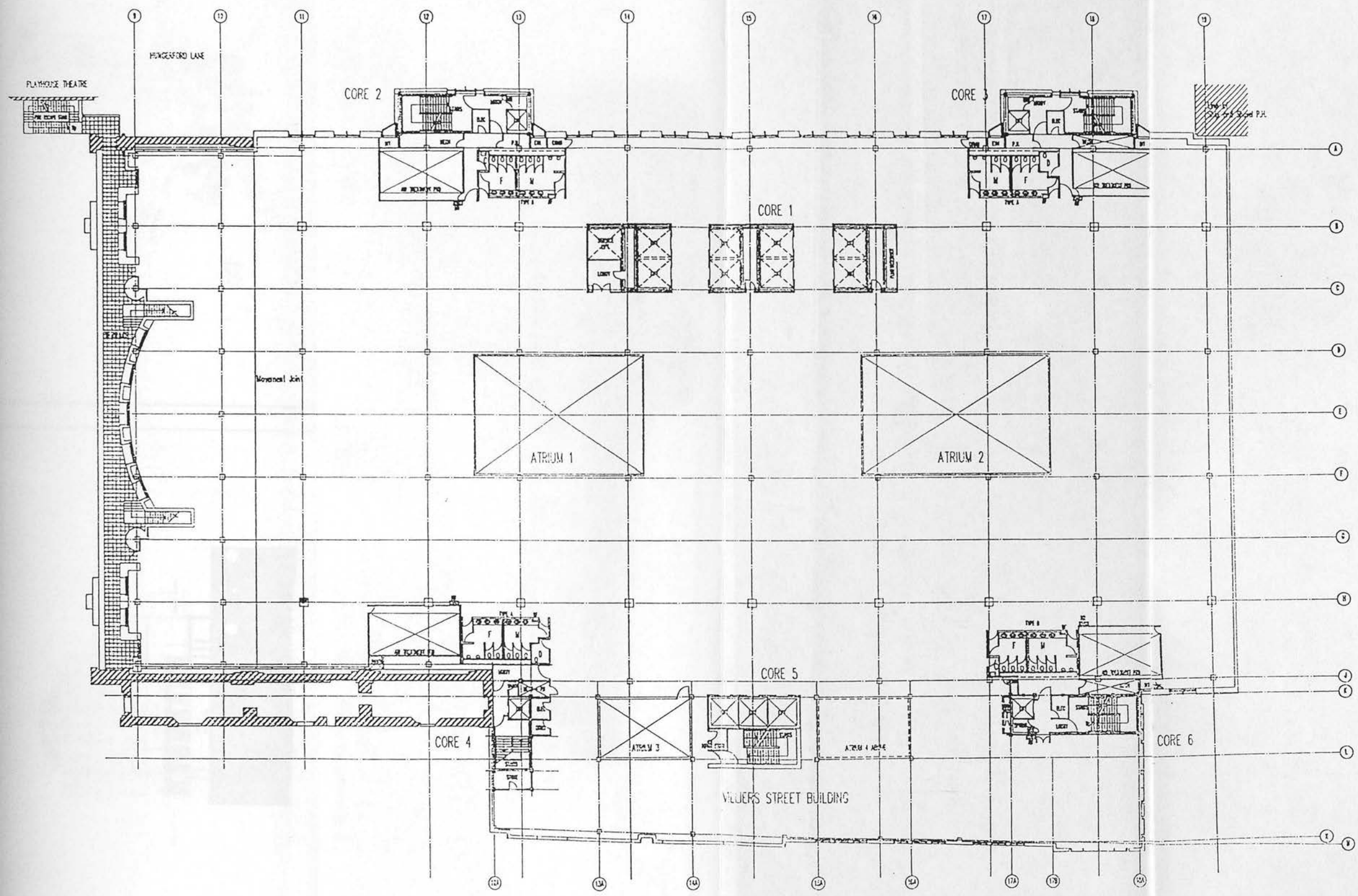
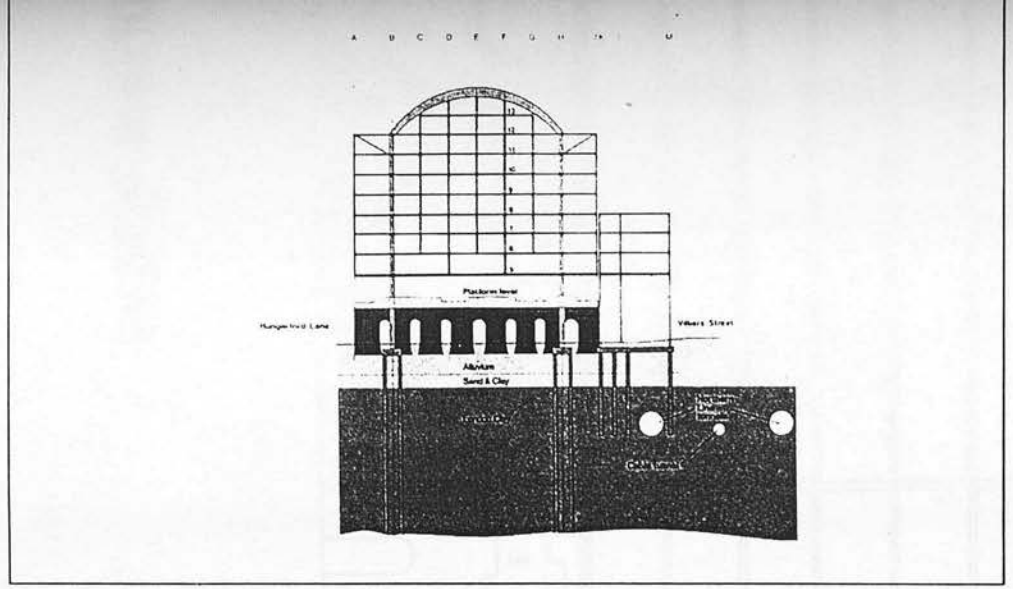
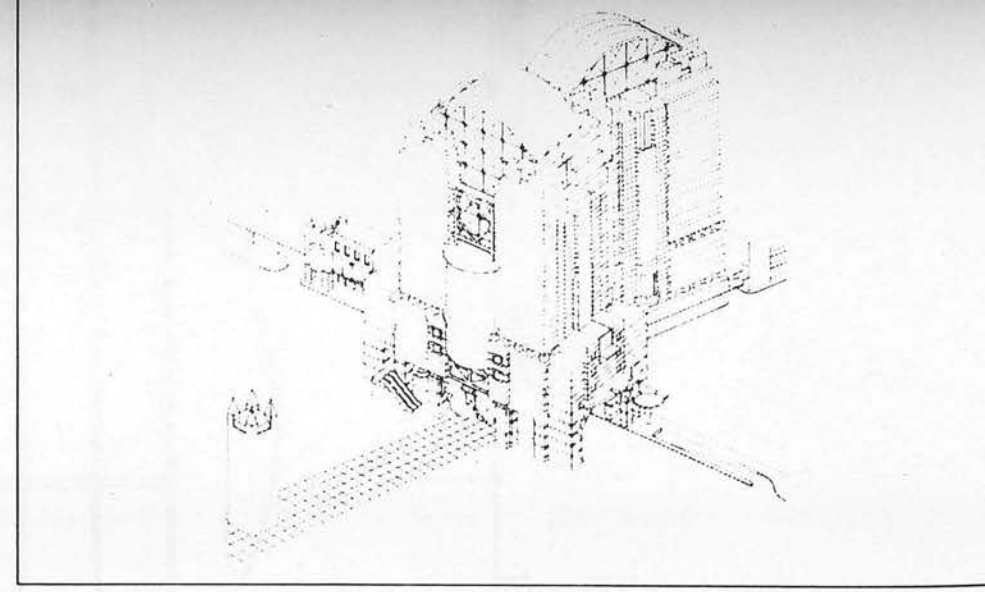


Fig 3B
EMBANKMENT PLACE
LEVEL 5
1:400 SCALE
20-07-88

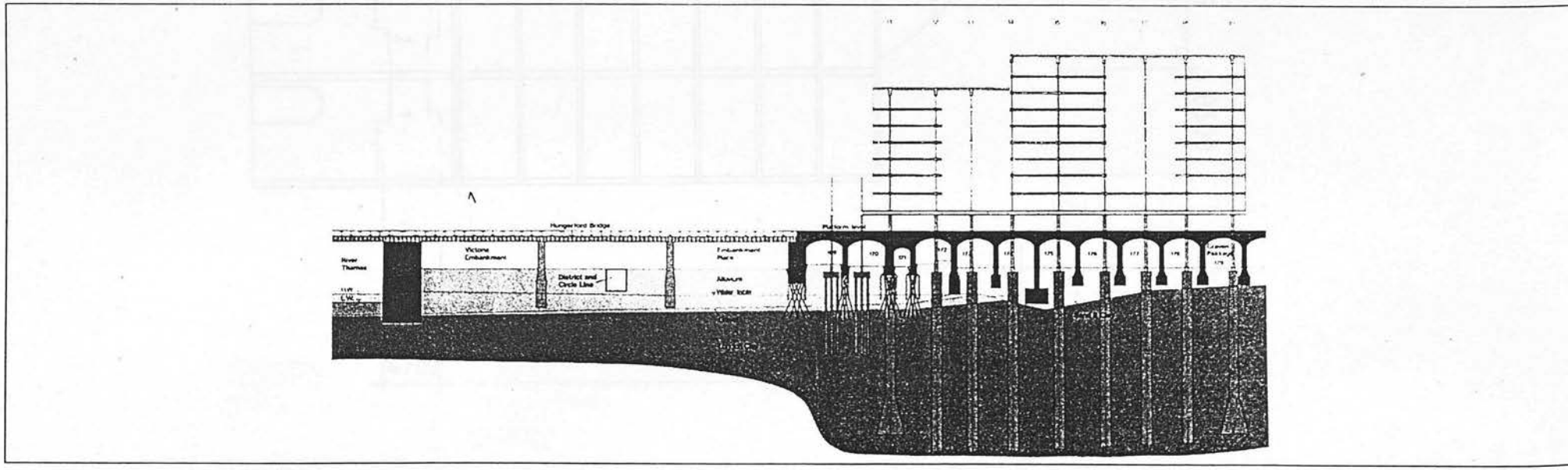


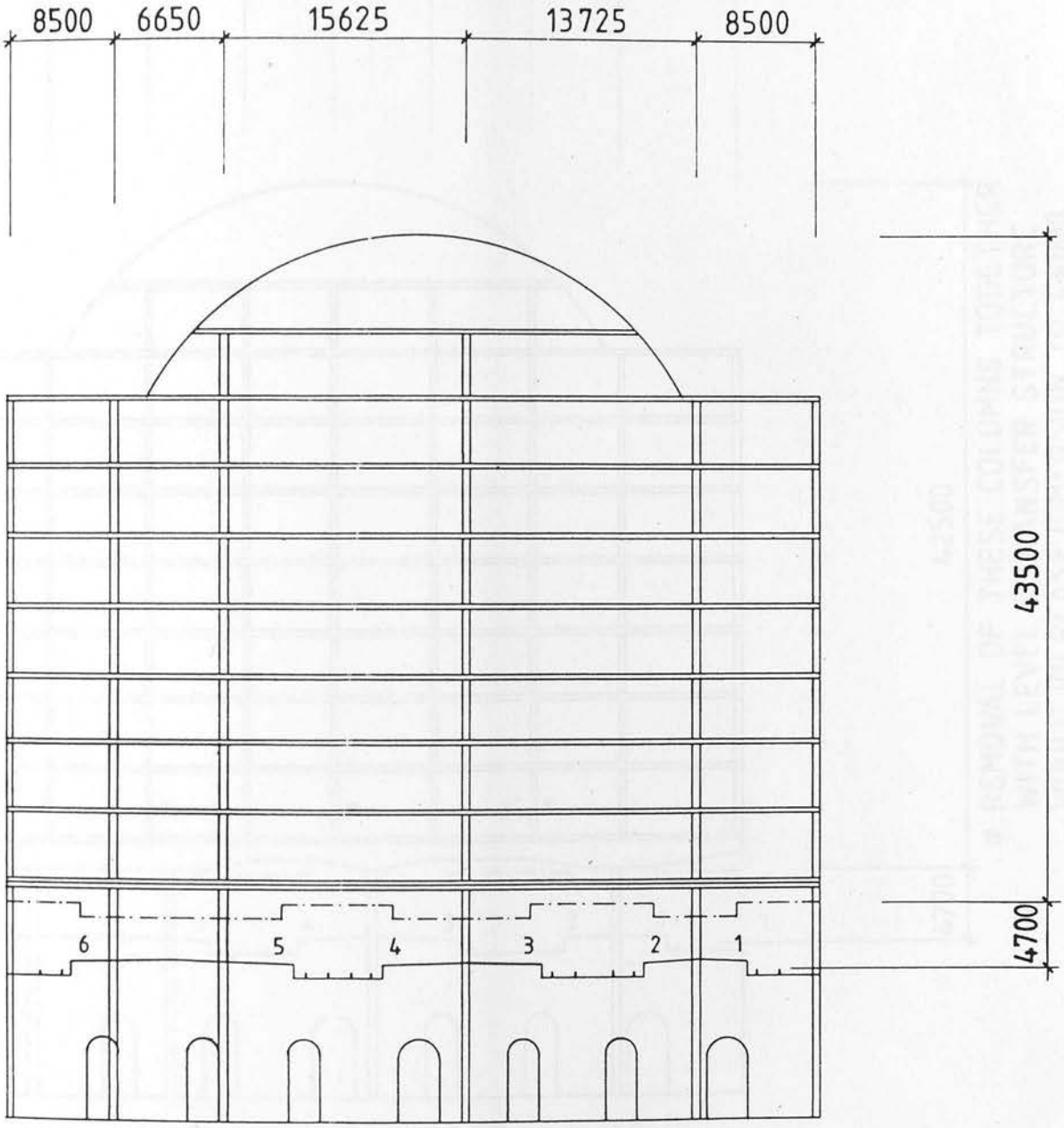
EMBANKMENT PLACE, CROSS SECTION

FIG 4

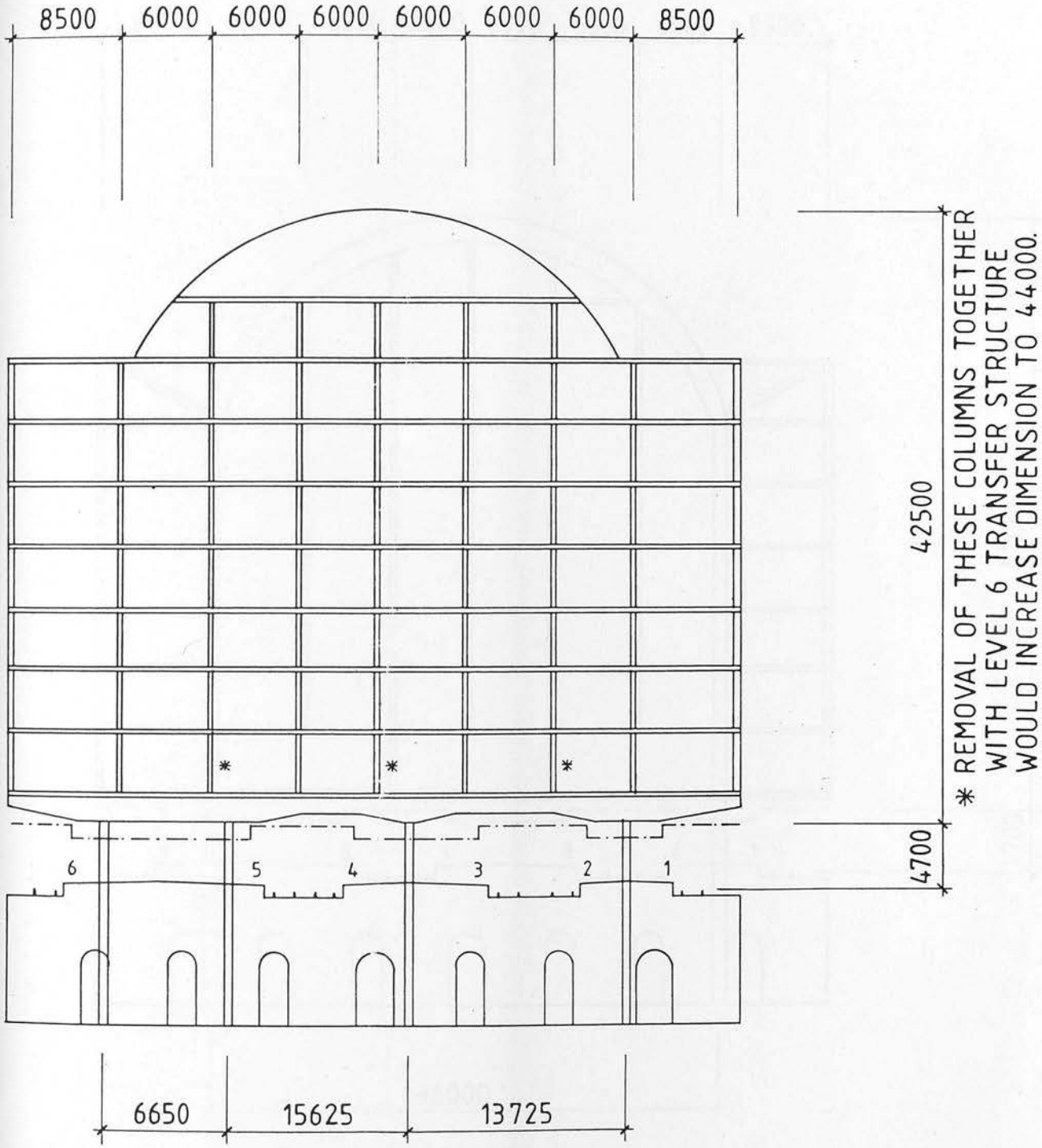


LEE HOUSE, AXONOMETRIC



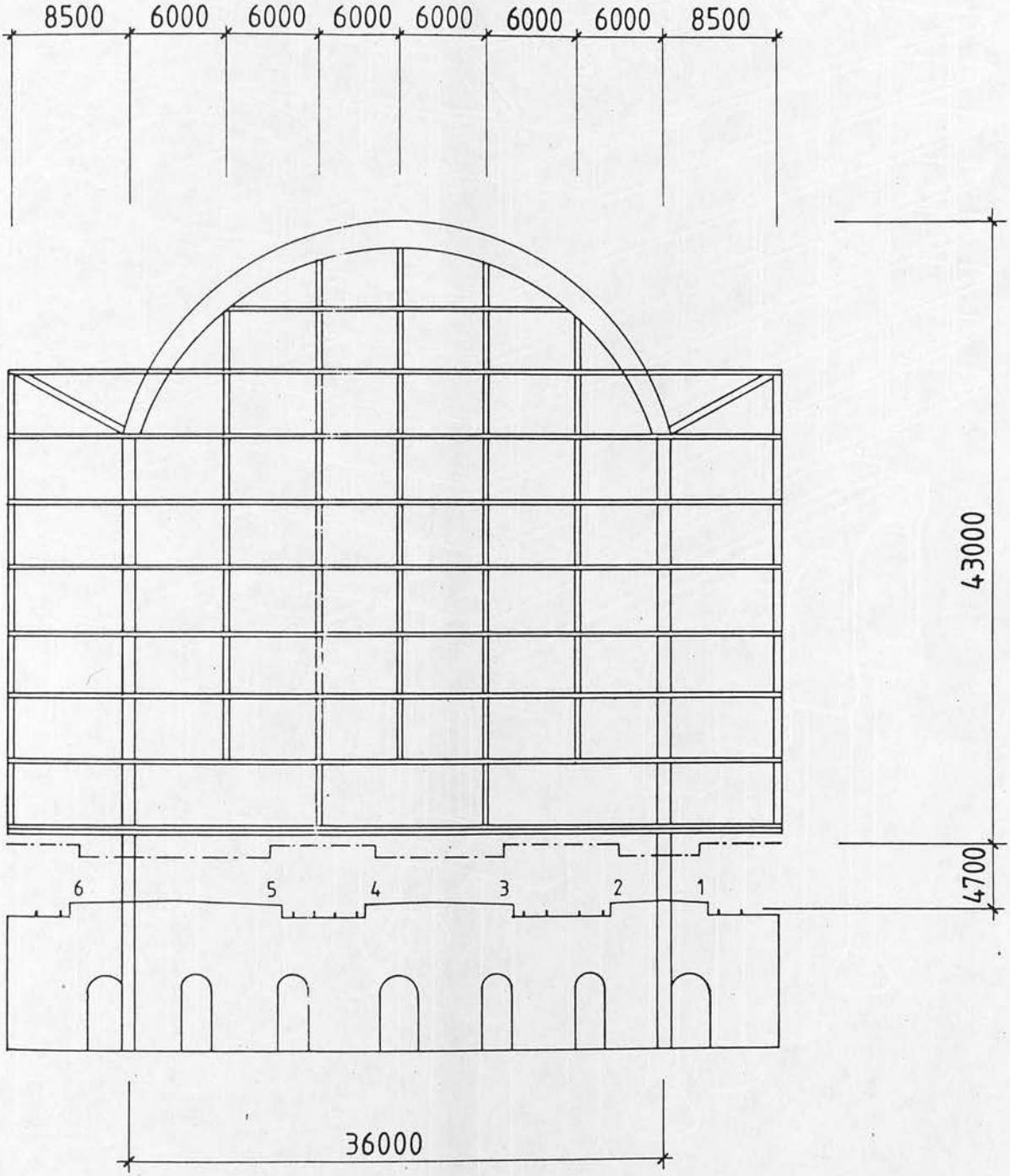


SIX COLUMN SCHEME
FIG 5A



UR COLUMN SCHEME

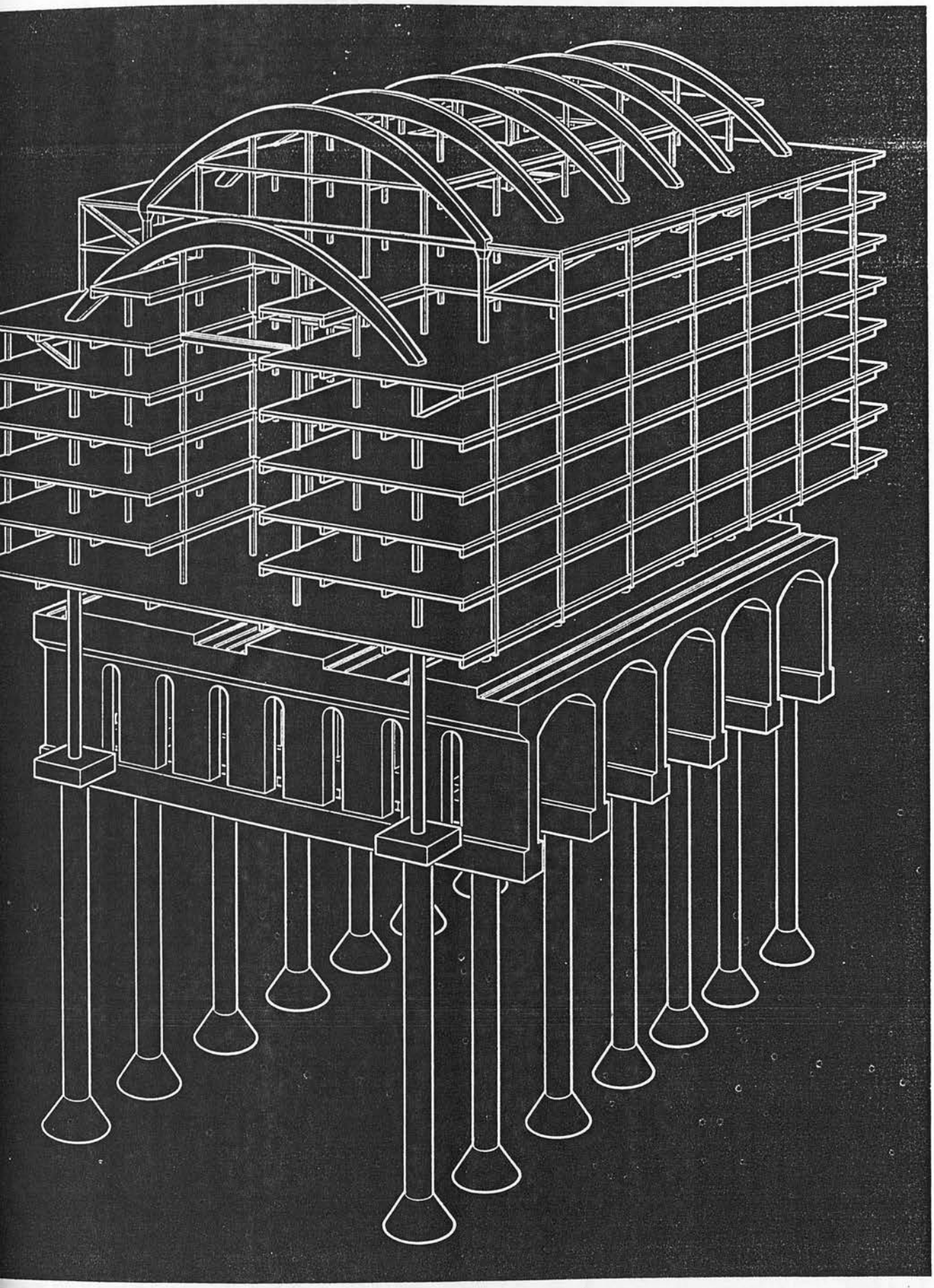
G 5B



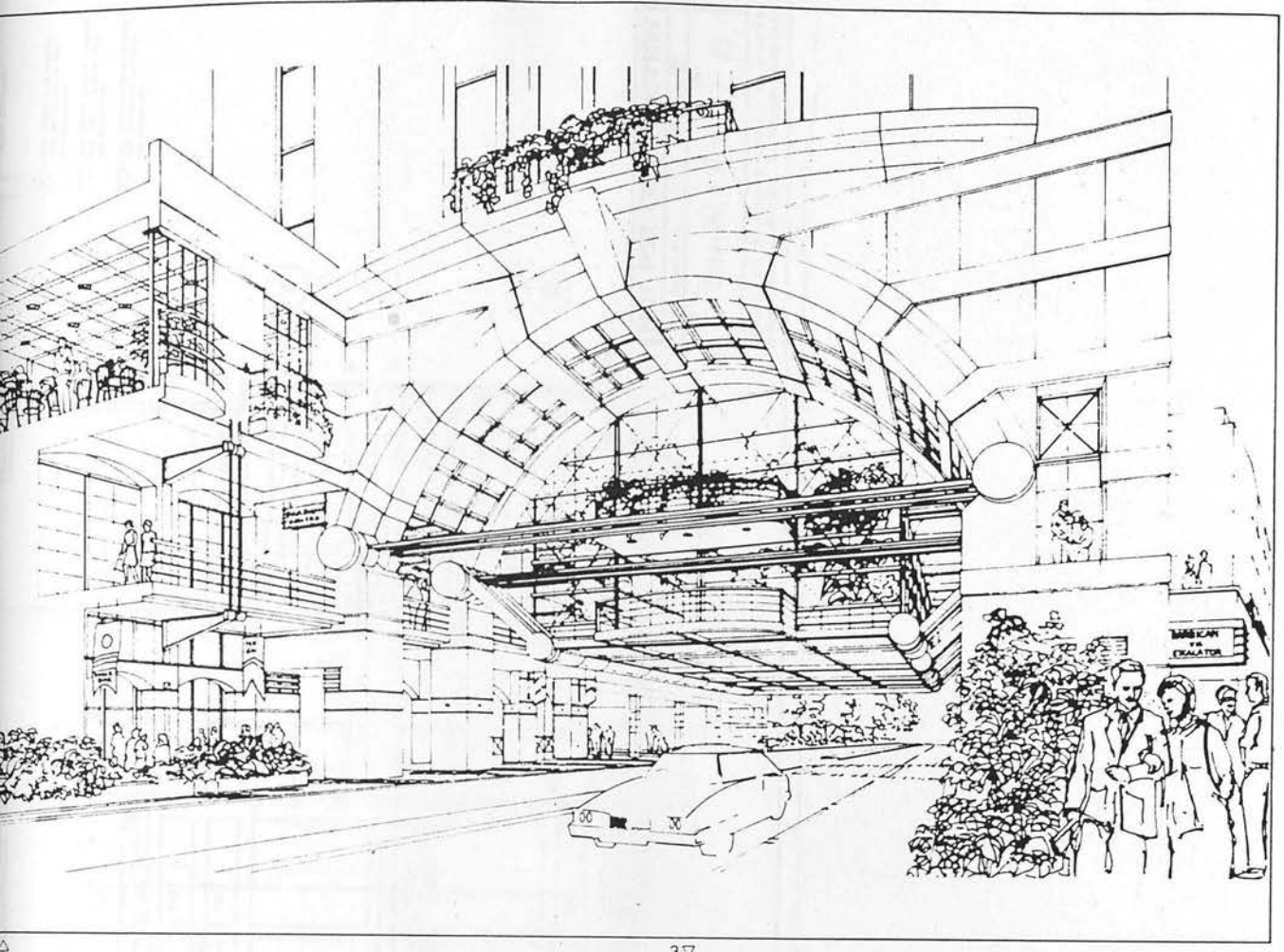
TWO COLUMN SCHEME

FIG 5C

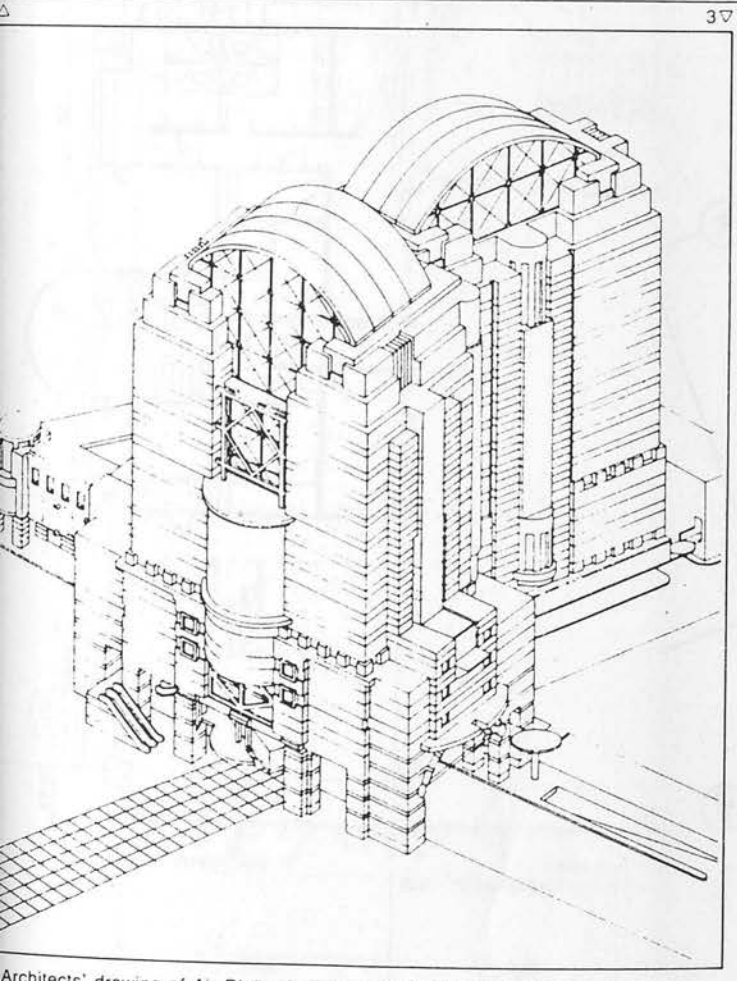
FIG 5D

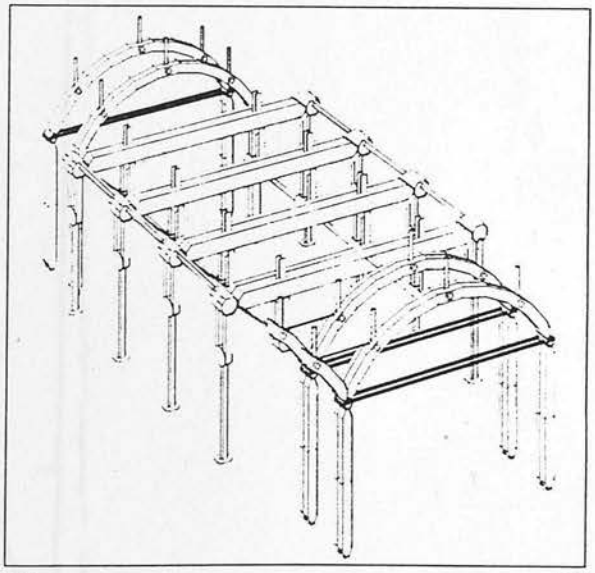


TERRY FARRELL PARTNERSHIP EMBANKMENT PLACE



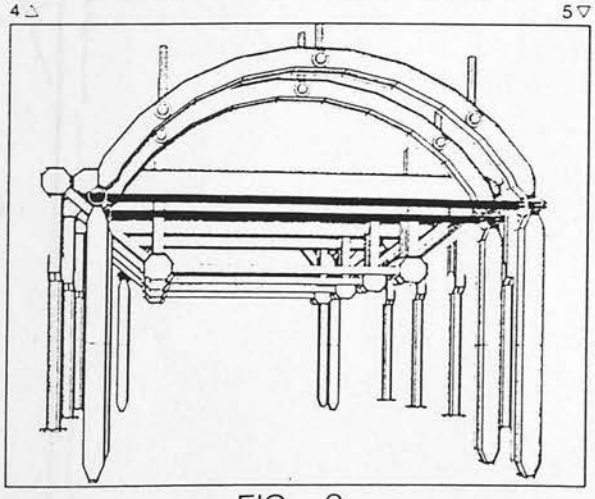
3▽





4△

5▽



Architects' drawing of Air Rights building and 3. View through archway
eproduced by courtesy of Terry Farrell Partnership)

Computer plot of transfer structure from the air and 5. From the road
aphics: Terence Haslett)

FIG 6

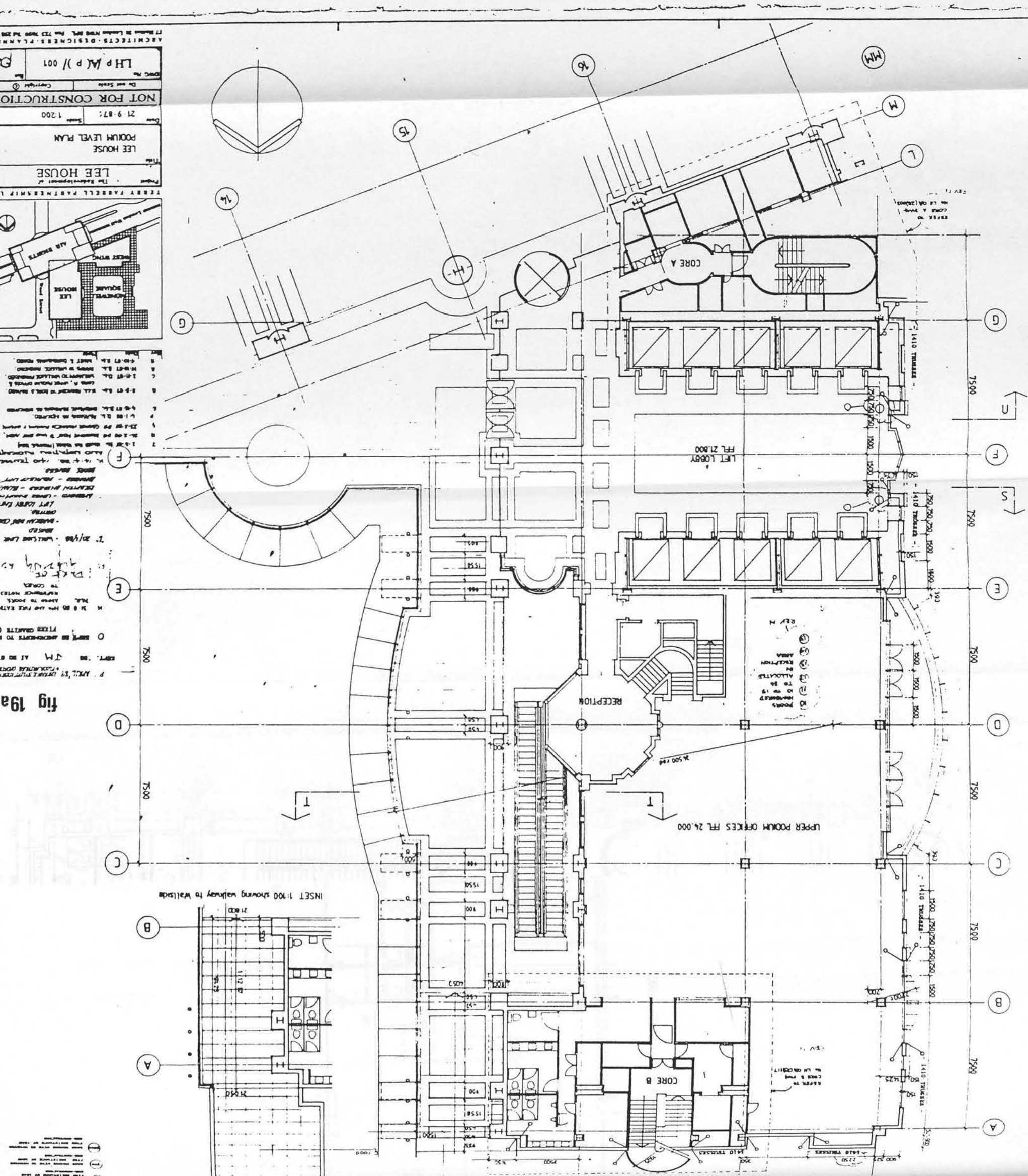
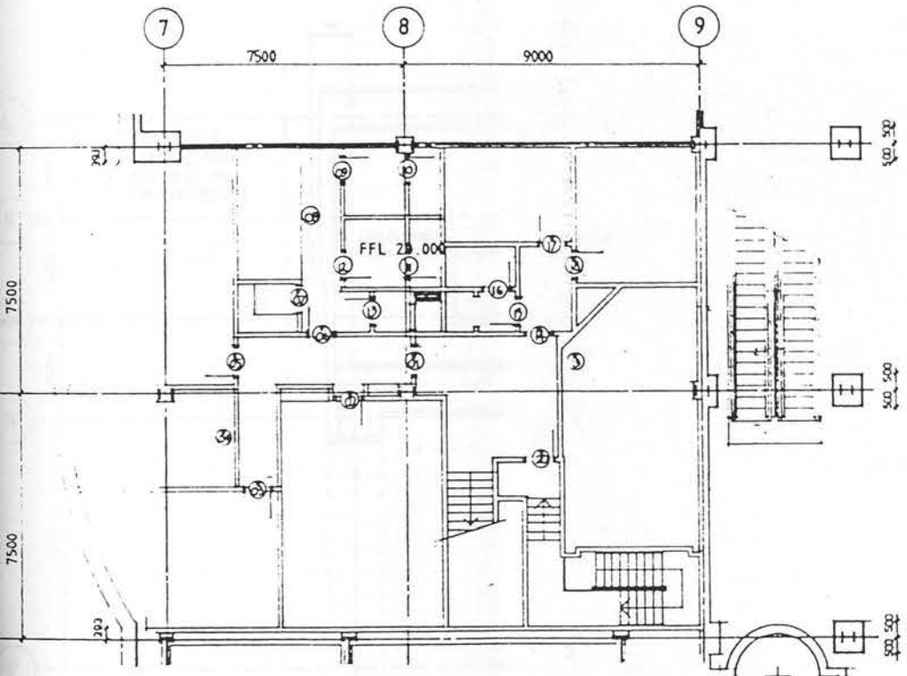

LEE HOUSE
POOLUM LEVEL PLAN
NOT FOR CONSTRUCTION
LIFT LOBBY
FFL 21.800
RECEPTION
UPPER POOLUM OFFICES FFL 24.000
CORE A
CORE B
INSET 1:100 showing walkway to waterside
7500

fig 19a



LOWER PODIUM PLAN 1:100

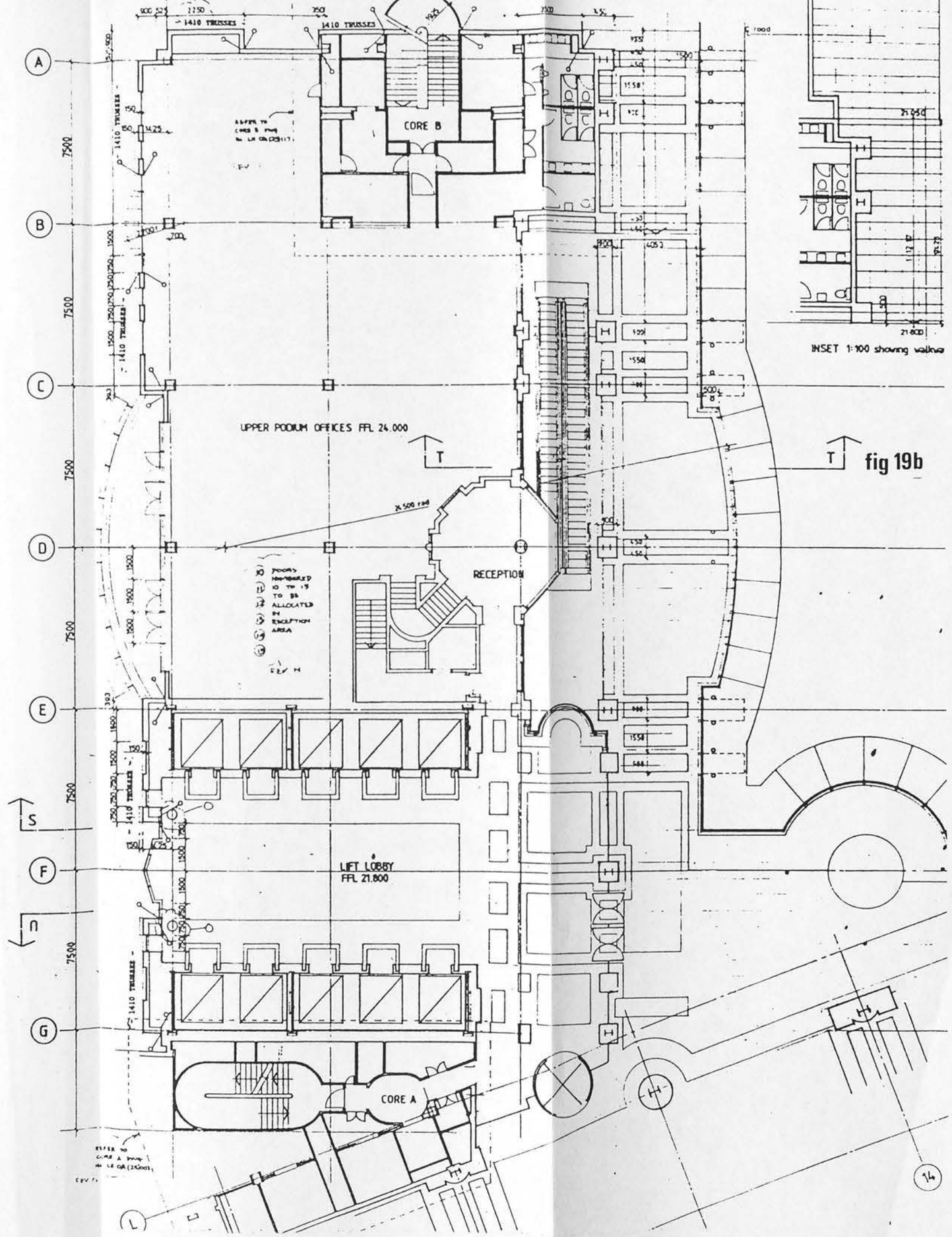
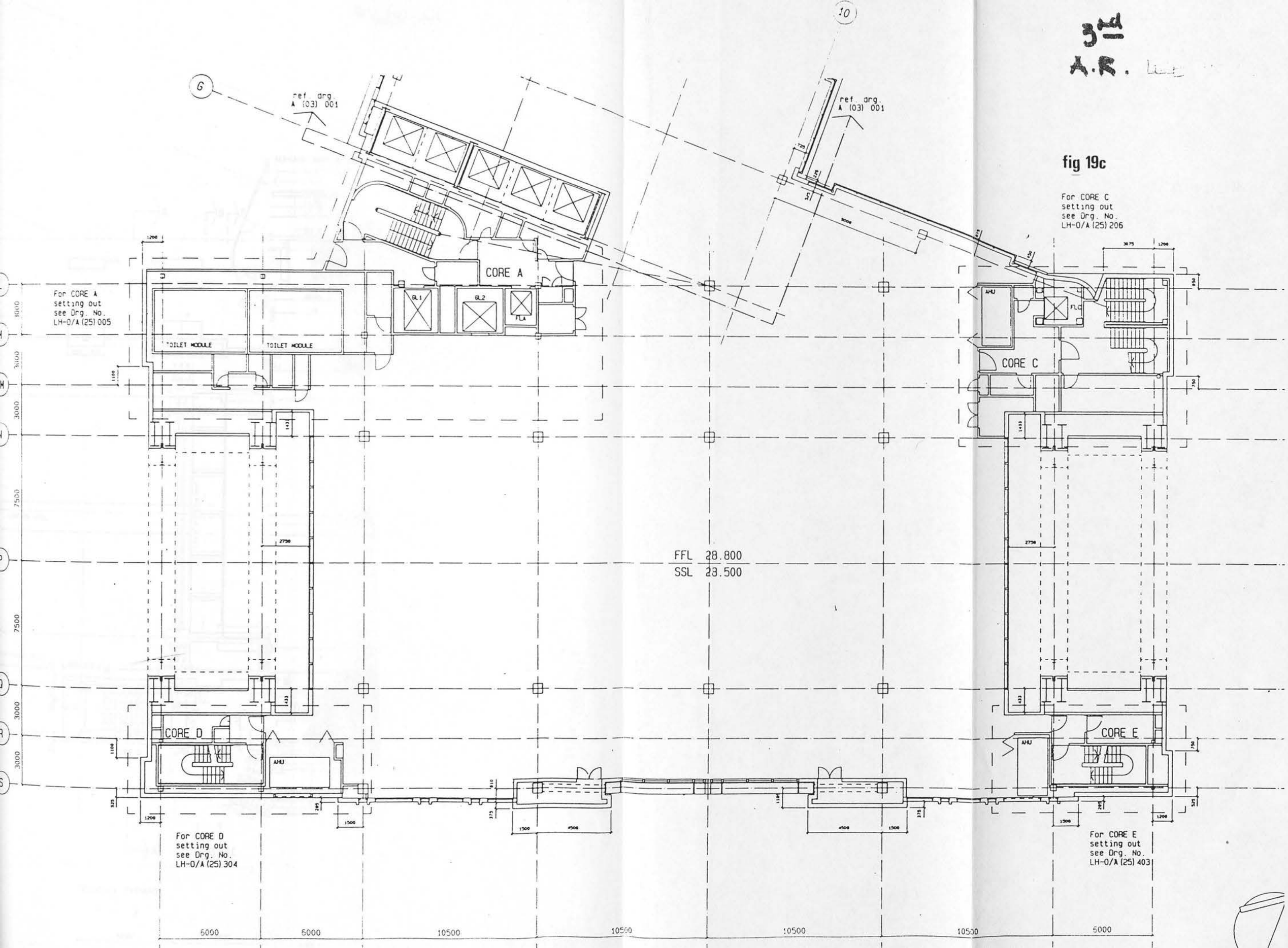


fig 19b

3rd
A.R.

fig 19c



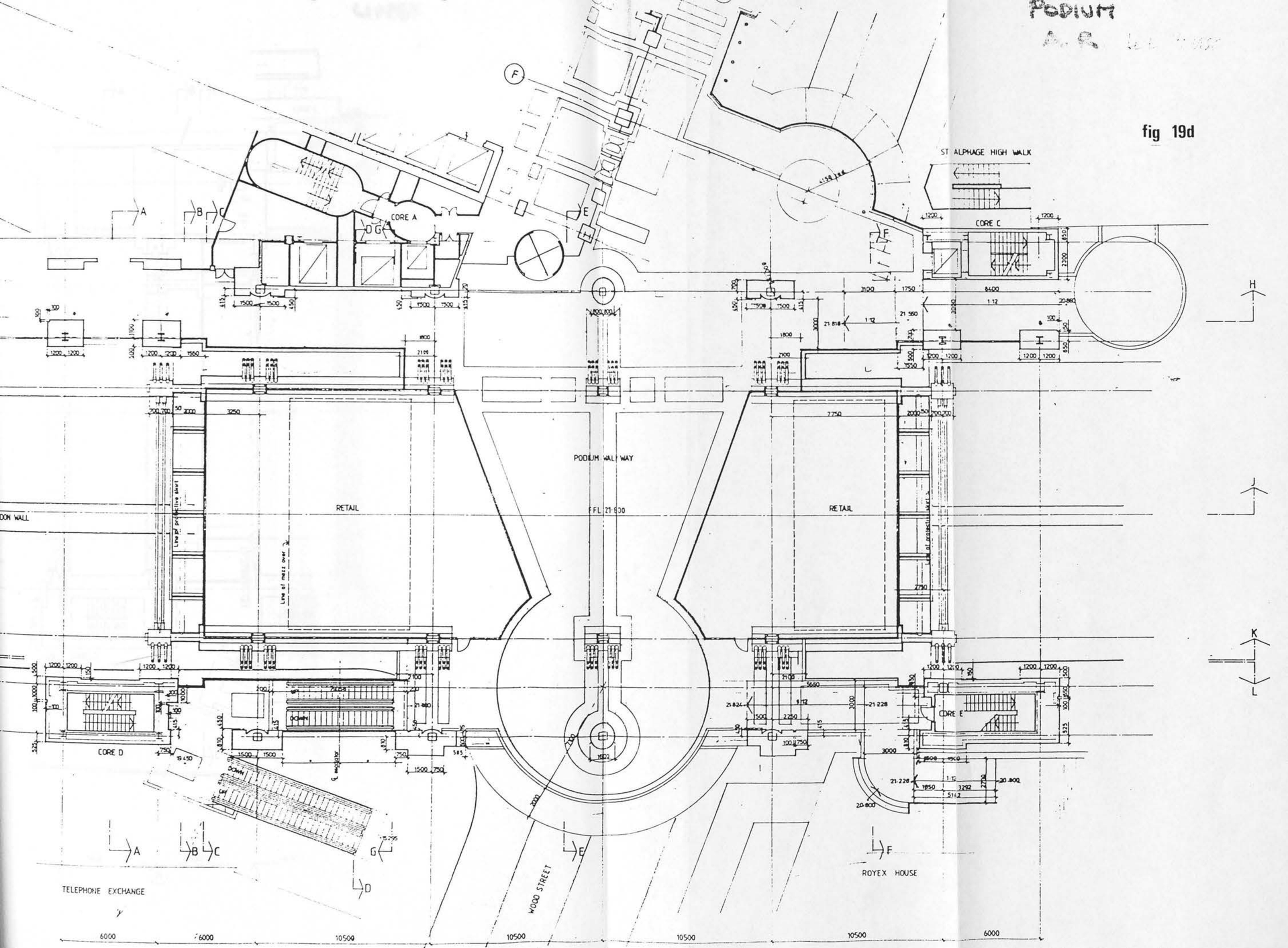
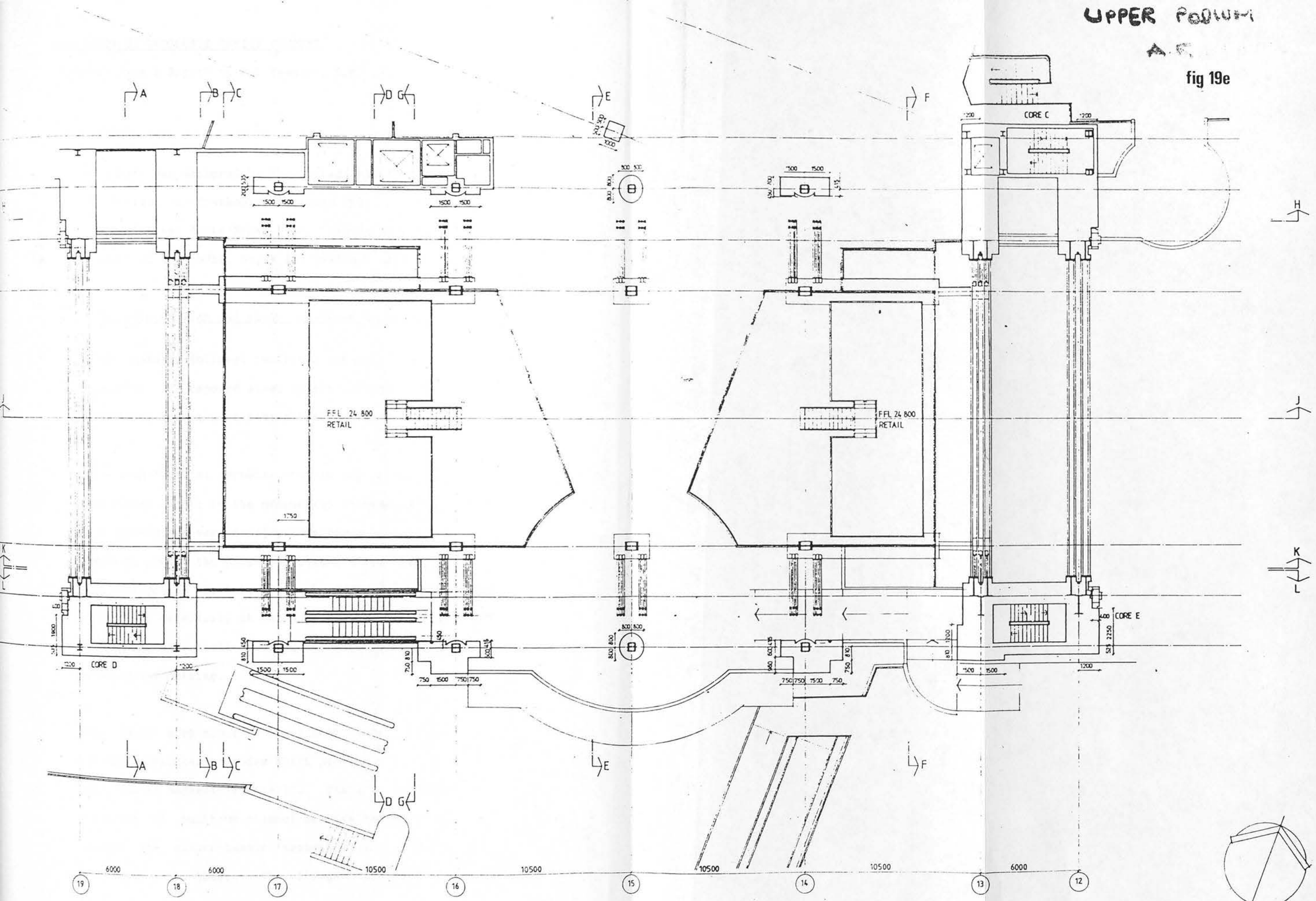

PODIUM
ST ALPHAGE HIGH WALK
CORE A
CORE C
CORE D
CORE E
PODIUM WALKWAY
RETAIL
RETAIL
FFL 21.800
WOOD STREET
ROYEX HOUSE
TELEPHONE EXCHANGE
6000
6000
10500
10500
10500
6000

fig 19d

UPPER PODIUM

A.5

fig 19e



CASE STUDY 5: BROADGATE OFFICE COMPLEX

(Excerpts from a Report by Hal Iyengar, SOM)

1.0 General

Broadgate was designed by the Chicago office of Skidmore, Owings and Merrill for Stanhope Properties PLC, the owner. The general contractor was Bovis/Schal, the fabricator-erector was the joint venture of Hollandia, Buyck and Smallman (HBS).

2.0 Architectural aims and their influence on structural articulation

Steel details followed two basic concepts: one pertained to the character of exposed steel on the exterior; the other to the simplicity and ease of fabrication and erection.

The architectural premise was to emphasise honest and clear structural logic in the proportion of members and joints while the aesthetics were based on expressing crisp, open, web-like forms to permit the play of daylight through the structure. This was integrated suitably with the expressions of robustness and integrity, especially at the joints. For ease of fabrication and erection, all field welding was eliminated in favour of shop welding and bolting.

The basic arch segments are linear elements with end-bearing plates connected to nodes which provide the angle change to the next linear segment (Figure 1). The arch members themselves are composed of built-up channel members arranged back-to-back to permit the column-hanger members to pass between them and be connected to them. The flanges of the channel provide

articulation and crispness to the otherwise solid arch shape. Regularly spaced batten plates tie the two channels together to make them function as one and provide a certain openness in the width of the members.

Architectural form, expression and articulation are all based on the exposed, painted steel structure for the Broadgate Project, a major office development on the north-east edge of London. the building enclosure forms a smooth metal and glass skin background to enhance the clarity of the structure. Member proportions and joint details follow strict structural logic to express directly the functions and workings of the structure.

The 10-storey office building, which is the focal point of the Broadgate Project, faces the Liverpool Street Station train shed, an historic structure of exposed iron and steel (Figure 2, 3, 4). Because this prominent position is also heavily congested, with tracks below, three important objectives in the design of the building were established: one, the structure should efficiently clear span over the tracks to provide unobstructed operations for the trains; two, the structure should be sympathetic to the historically significant train shed; and three, the building should act as a centerpiece whose articulated expression would contrast with the neighbouring complex of stone and glass clad buildings.

The office building, with approximately 550,000 gross sq ft of office and trading-type space, is supported on four segmented, tied parabolic arches spanning the 256 ft over the railroad tracks. The two exterior arches, their ties and the columns and

beams that frame into them, are located so as to create a gallery at the perimeter which permits the exterior structure to be exposed, creating a structural expression for the building. Member prroportions and joint details followed strict structural logic to express directly the functions and workings of the structure.

3.0 Options to the choice of the structure

The arch solution was selected from three possibilities. One alternative involved a traditional, cross-braced truss, seven stories tall, which not only involved 33% more steel, but also did not create an exciting architecture - and failed to relate to the historic station archways. Another alternate was a parabolic suspension system with end pylons similar to the Federal Reserve Bank structure in Minneapolis. This solution, though efficient as a structure, posed co-ordination and erection difficulties, it was felt that facade thus created lacked the "architectural definition" required by the designers. ~~definition of the facades~~.

4.0 Description of structure

The primary elements of the system are the four parallel, 7-storey high, parabolic tied arches which span 256 ft between the concrete buttresses. The two exterior arches are exposed and are set out from the fire-rated cladding (Figure 5). The two interior arches traverse through the body of the building and are partially expressed internally through atriums. The four arches form three bays, perpendicular to the arches, which are spanned by composite floor trusses. Vertical, exposed end trusses are provided in the middle bay to provide lateral stiffness for the

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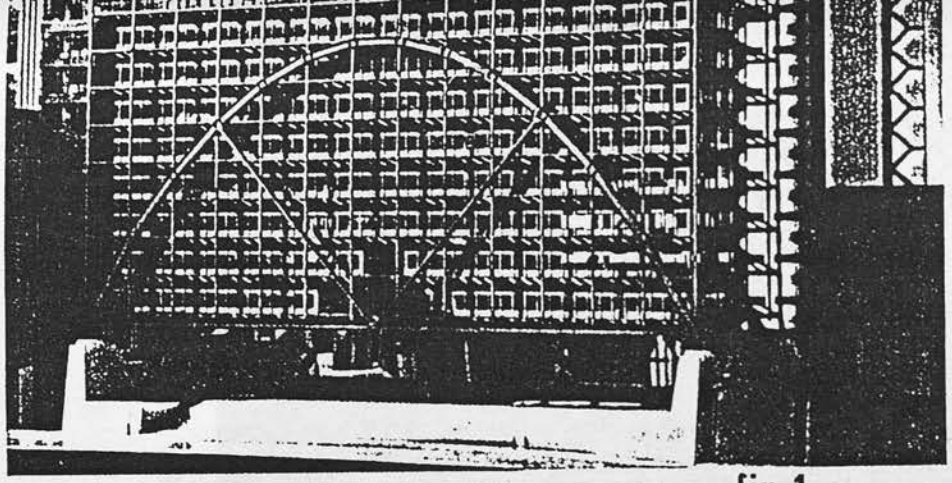
broad side wind forces and for lateral stability of the arch system.

Vertical hanger/column elements are supported on the arches at node points, and the floor framing members are connected to the hangar/columns through a typical shear connection. The gravity load flow then occurs from the floor trusses to the column-hangers, to the arches and to their supports.

At each floor level, a continuous floor girder is provided in the plane of the arch on the interior, which together with the arch provides for lateral stiffness for the entire building in the direction of the arch. These girders also function as intermediate ties. On the exterior arch, these intermediate ties are moved behind the cladding line and activated by diagonal struts on certain floors in the horizontal plane at the arch nodes.

The straight-segmented, parabolic shape was chosen for the arch as the most efficient shape for the primary loading configuration, a series of approximately equal point loads applied to the arch by the columns and hangars. The arch shapes matches the moment diagram for uniform point loads and therefore the loads are carried as axial compression loads with a minimum of flexural bending, down the arch to the buttressed walls.



Model of 10-story office building, centerpiece of Bishopsgate project **fig 1**

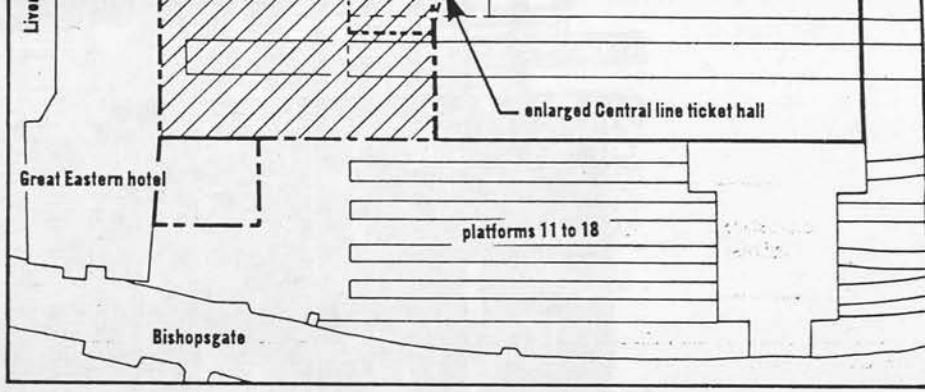


fig 2

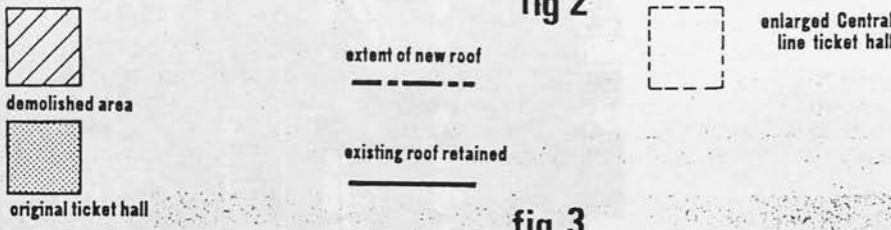
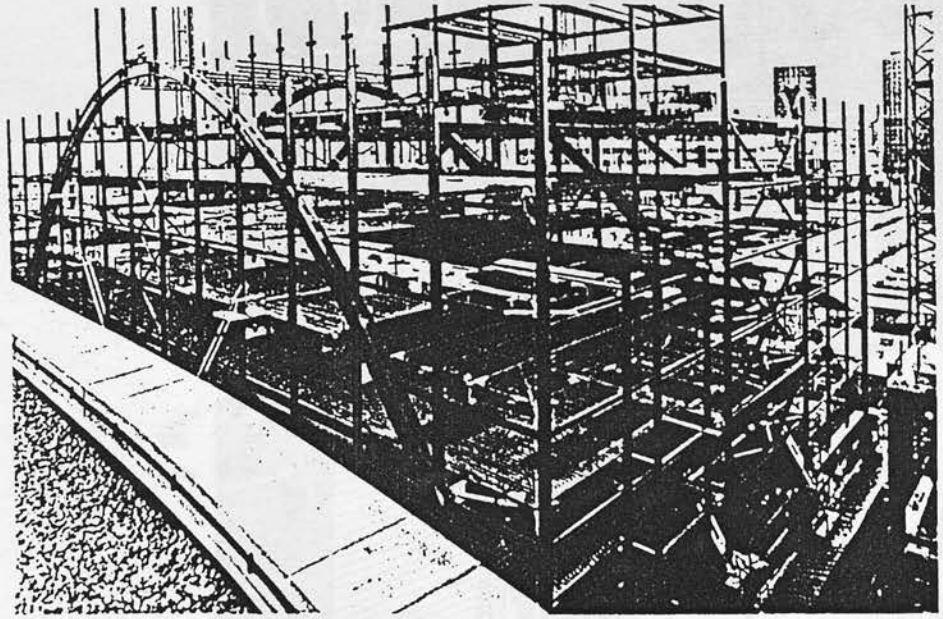
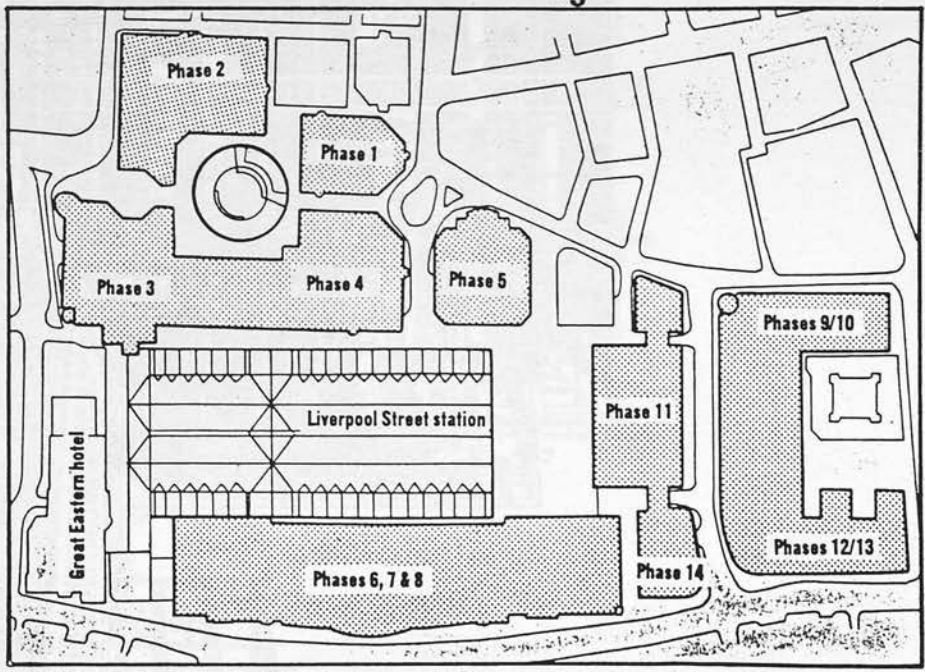


fig 3



Unique structure required high degree of dimensional control in steel fabrication. **fig 5**



ABOVE: Broadgate phases 6 to 14 are built over rail tracks into Liverpool Street.

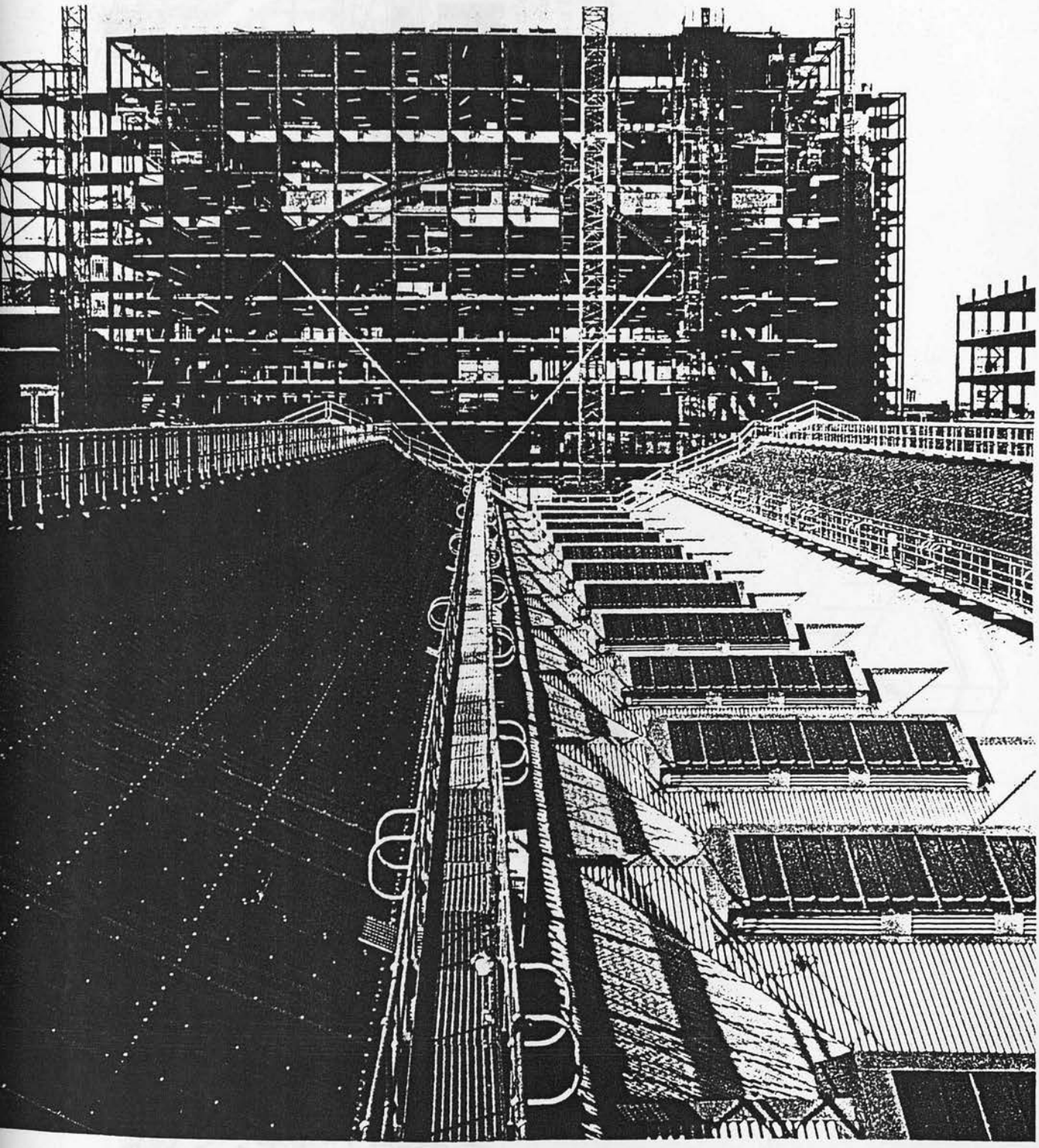


fig 4

CASE STUDY No:6

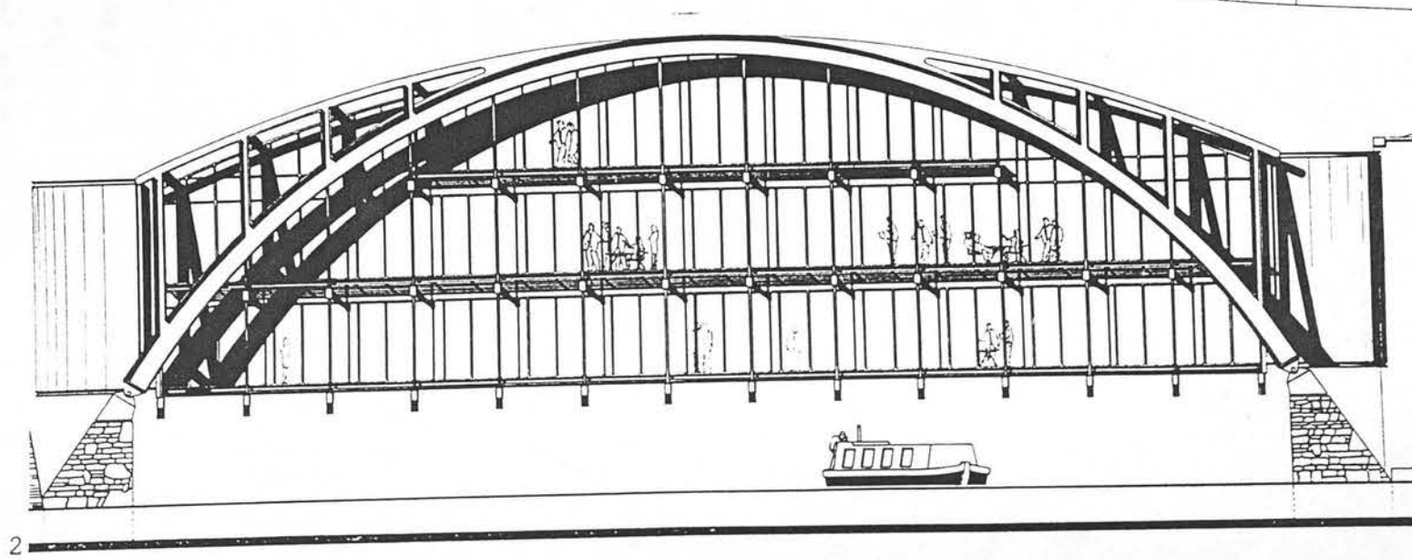
DAVID MARKS, JULIA BARFIELD ARCHITECTS

Practice set up in 1988 by DAVID MARKS (b1952), worked Richard Rogers & Partners 1980-88, latterly on Lloyd's; and JULIA BARFIELD (b1952), worked Richard Rogers & Partners 1980-82; then for Foster Associates 1982-88 on Renault, the BBC, and the Royal Academy of Arts diploma galleries among other projects.

Won first prize in 'Bridge of the Future' ideas competition run by the *New Civil Engineer* (with Jane Wernick of Ove Arup & Partners as consultant structural engineer) and placed joint third in Glasgow Eurodrome competition promoted by Institution of Structural Engineers (with Nick Maclean and Kelvin Hindson as consultant structural engineers) in 1988, but on leaving full-time employment 'every job disappeared'.

Current work includes scheme for redevelopment and refurbishment at Battlebridge basin, Islington (planning consent now being negotiated) and feasibility proposals for regeneration at La Ciotat, near Marseilles, in anticipation of naval dockyard closure (negotiations with national and local authorities now in progress).

1



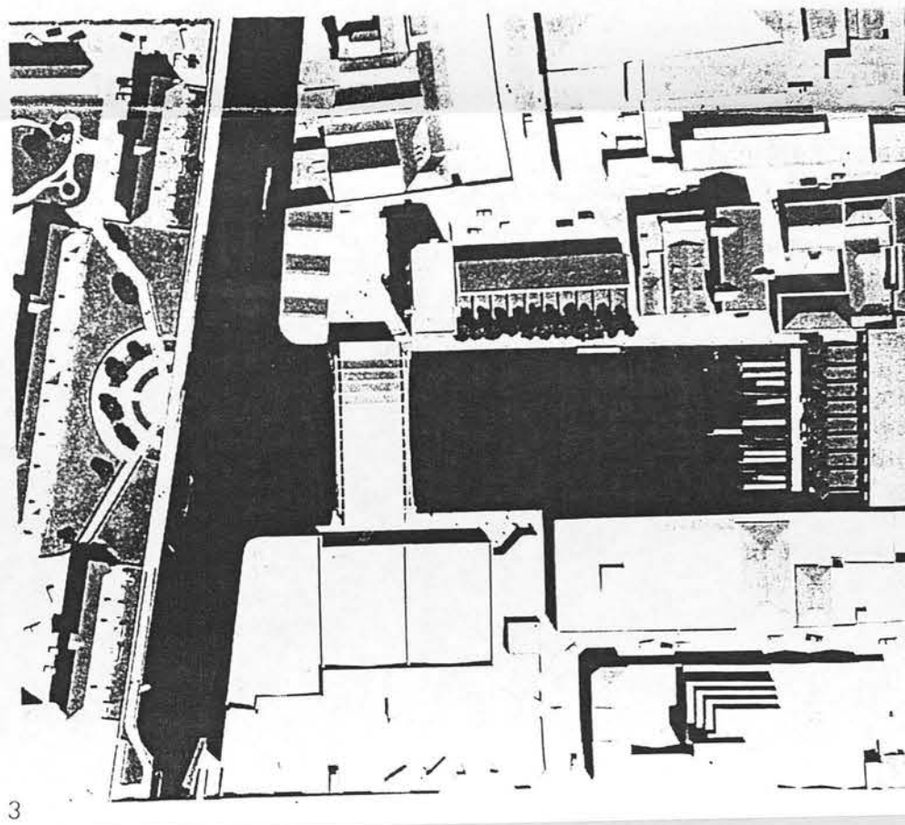
2

current project 1

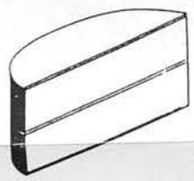
Battlebridge basin, for London Buildings plc

Creation of some 85 000 sq ft office/business space and canal museum by refurbishing existing buildings and providing new ones—in the form of a suspended bridge structure over the entrance to Battlebridge basin and of a new range of four-storey buildings overlooking the Grand Union Canal, together with the provision of new pedestrian access ways round the canal basin.

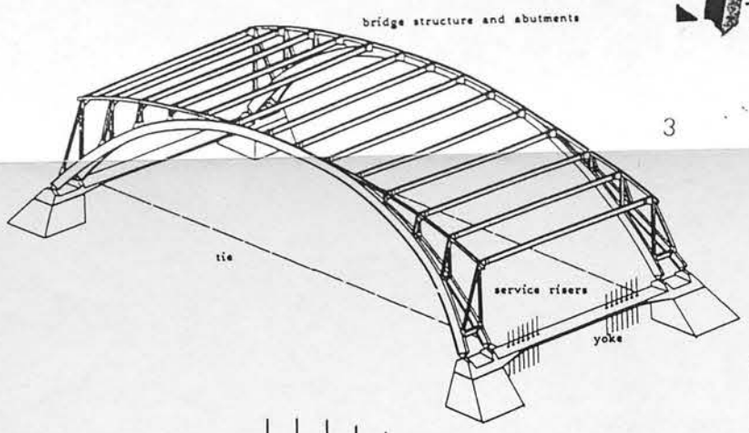
Structural engineers: Ove Arup & Partners

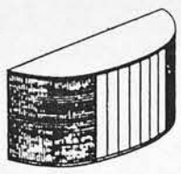


3

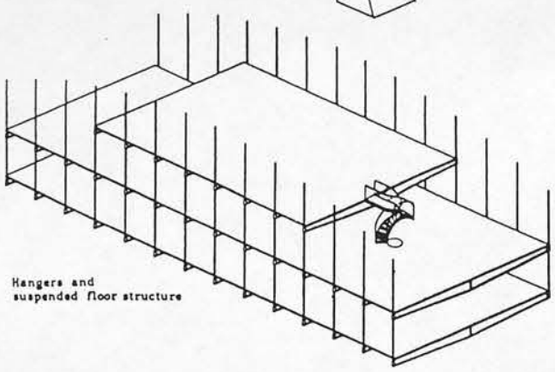


ancillary services





vertical circulation



Hangers and suspended floor structure

1, 2, the Battlebridge scheme, in w offices are slung across the mouth the dock in a bridge-like structure 3, the urban connection. 4, structural system.

CASE STUDY 7: THE EXTENSION TO THE IMPERIAL WAR MUSEUM

THE IMPERIAL WAR MUSEUM (Barrel Vault Roof)

Introduction

The aim of the extension to the existing museum building at Southwark built by James Lewis in 1815 was to provide more exhibition space with room for study galleries and improved administration areas. The original building functioned as the Bethlem royal Hospital and the Imperial War Museum took over the building in 1936. It is dedicated to exhibits of the two World Wars and subsequent events involving the British armed forces.

Site Context

The existing building is designed in the classical style with symmetrical layouts in plan and elevation. The building is sited on a flat park ground (Figure 1A).

Architectural Concept

The primary concept was for the creation of a large and memorable central space achieved witthin the envelope of the existing building by the infill of an existing courtyard (Figure 1B). The main exhibition space is at ground level and above this, on three sides are two levels of galleries which house the smaller exhibits. The plant rooms are housed above the upper level of the galleries. The barrel vault runs the length of the central space and provides the means whereby the central space is lit. It also provides a strong visual link between them like a continuous spine through the length of the building. The supporting structure was articulated for this purpose.

Various overall planning options for the courtyard area were studied and described in Figure 1B. Option D was selected as the best solution to the architectural, functional and economical requirements considered.

Constraints

The heaviest exhibits were to be housed on the two lower levels, level A and B (Figure 2A) whilst the upper two levels, C and D comprised galleries. Services would be concealed in the ceiling space peripheral to the roof vault. The central area has a clear span of 22.7 m with a maximum height of 23.4 m to the crown of the barrel vault. This will be used for large exhibits such as aeroplanes and rockets. The problem comes with the suspension of aeroplanes within the atrium:

a) They represent point loads which will have to be adequately resisted by the structure without restriction to layout and positioning.

b) The wing spans get in the way of vertical supports to the building structure.

Development of Structure

The structure which developed out of constraints (a) and (b) also supported the barrel vault roof, and it had to resist the horizontal thrusts of the arch frames used in the barrel vault. The framing pattern of the vault itself responded to a separate set of criteria.

Figure 2B shows the cantilever jib frame which projects 5.5 m from its column to support the barrel vault roof at its springing points. The central space is 23 m x 40 m long and 23 m high at the top of the barrel. The jib frames are spaced at 7 m centres and aircraft are suspended from lugs welded to the construction nodes between vault edge and jib frame.

Horizontal restraint was achieved by transferring out-of-balance forces through the secondary beams into the in-situ composite floor slab which acted as a horizontal diaphragm between four cantilever rc ducts situated one in each corner of the central space (Figure 2C). The use of twin tubular columns was an aesthetic response to reducing the visual impact of the structure whilst emphasising the exhibits; it was also considered to be visually appropriate to the exhibits of war machinery. The idea of light structural steel framing is carried through in the framing pattern of the barrel vault. Here, several options were studied before deciding upon the two-way diagonal lattice grid which was visually most acceptable in meeting the criteria of accentuating the linear atrium (Figure 1C).

The decisions to suspend the exhibits from the cantilever jib frame and not from a barrel vault that spanned the full distance between the vertical supports were made on the following grounds (Figure 4).

a) The increase in span would imply the need for larger-sectioned members necessary to cope with the increased dead loads of the vault.

b) The additional imposed loads from the weight of the exhibits would also have the same effect as (a) whilst restricting the layout of the exhibits because point loads to arches have to be

kept as uniform as possible, or conversely, the exact locations of these loads would have to be designed for in the framing pattern of the vault, and it becomes difficult to rearrange the exhibition layout, if required.

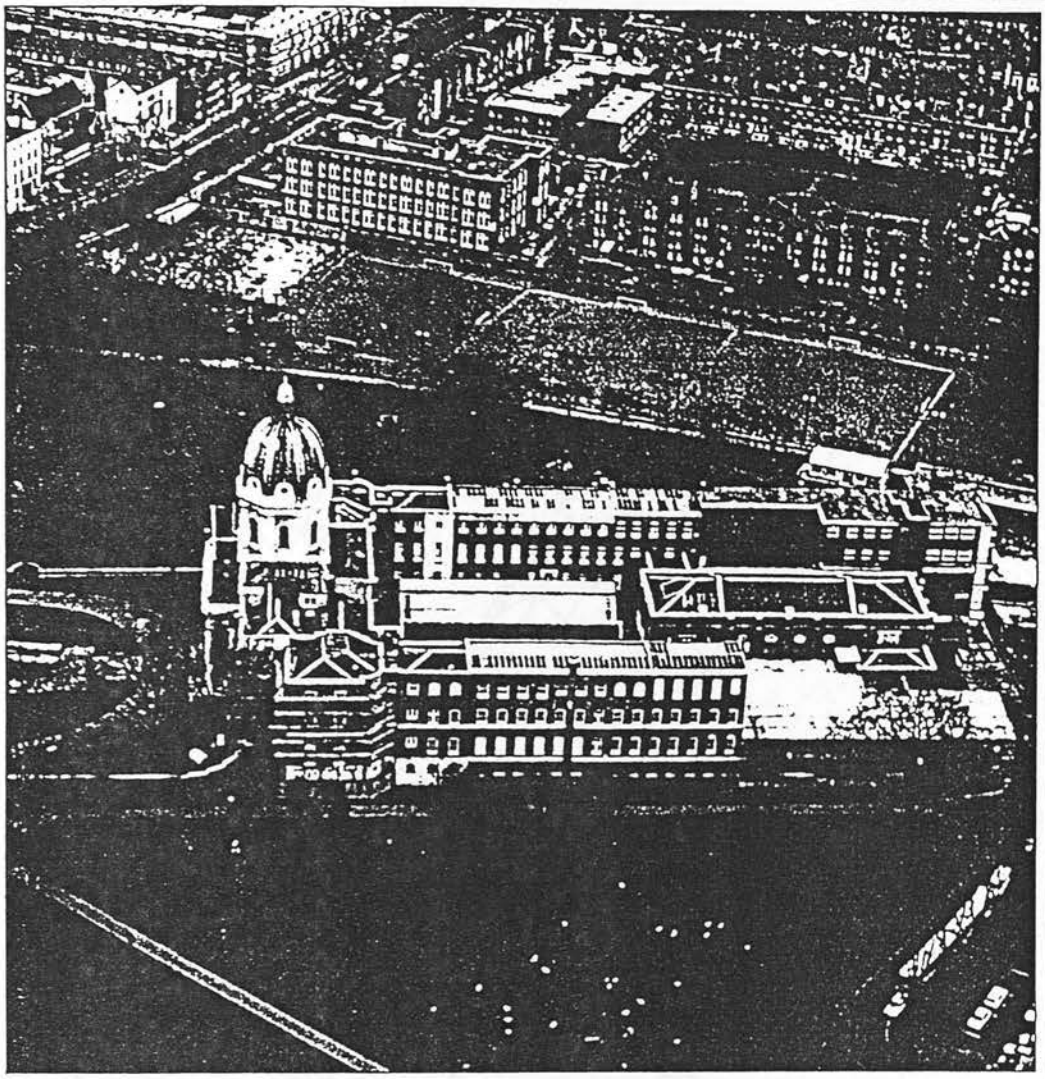
It must be noted that the solution itself posed some constraints:

Five aircraft were suspended in various positions and configurations from the edge beams of the barrel vault, and modifications to the aircraft were needed, their centres of gravity located and positioned in space with a 1:20 scale model of the museum courtyard. Steel cables used for the suspension were established with criteria for maximum and minimum angles and the effects of these loads were anticipated in the analysis and design of both cantilever frame and roof vault.

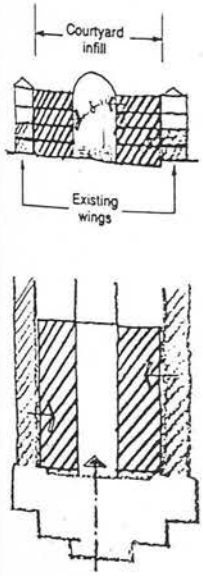
c) The permissible deflection of the roof vaault might have been exceeded if it were loaded with exhibits.

The aerial photograph shows the Imperial War Museum prior to the construction of the extension

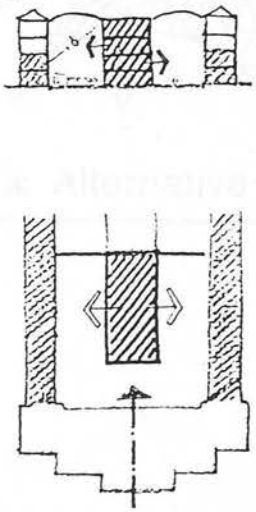
fig 1a



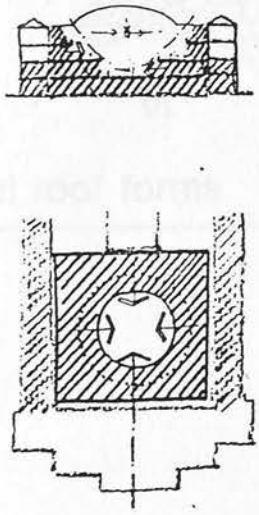
Option A
Central space flanked by four levels of gallery accommodation. The central open space is restrictive for large exhibits.



Option B
Accommodation in the central area with open galleries on both sides. The central plan does not maximize the area of gallery space within the courtyard.



Option C
Whilst providing maximum gross area the plan imposes a strong, finite, 'circular' form.



Option D
A large open space with surrounding tiered galleries was a favoured plan form on which further studies were based.

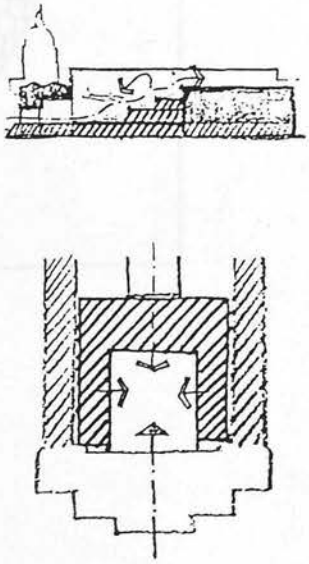
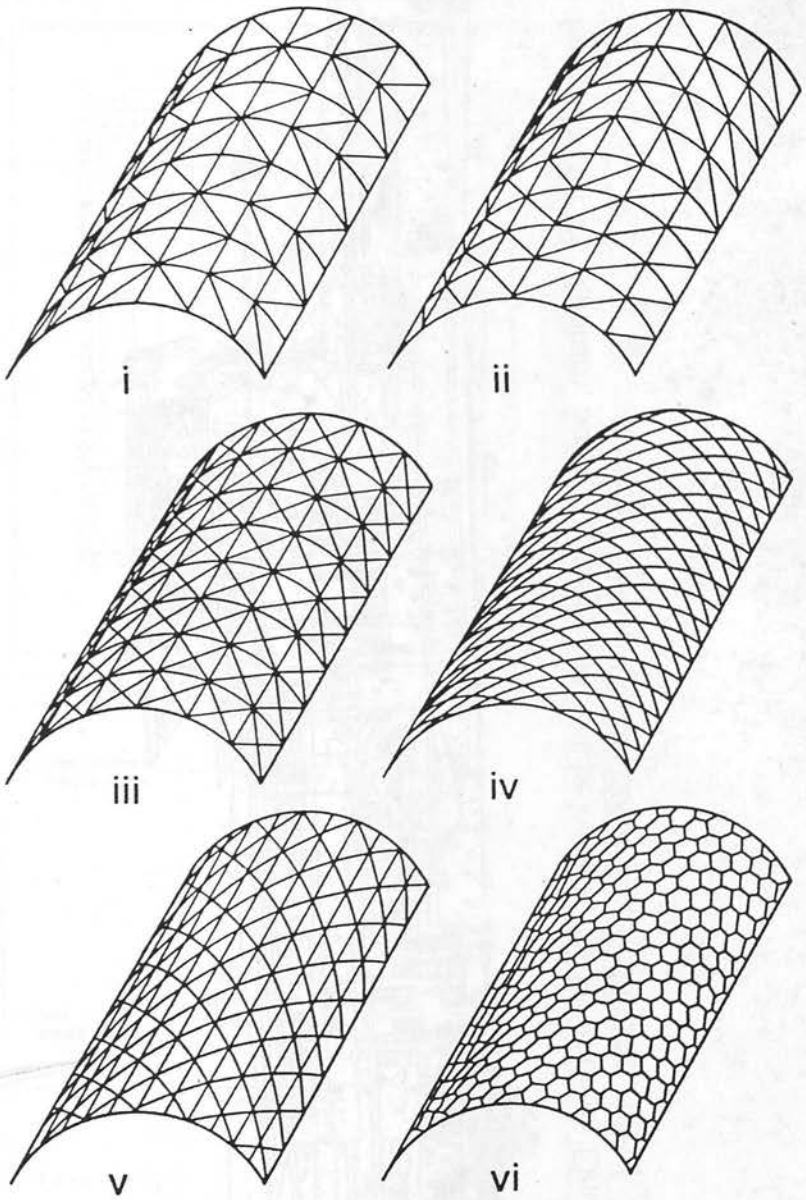


fig 1b
Planning options



1c Alternative barrel roof forms

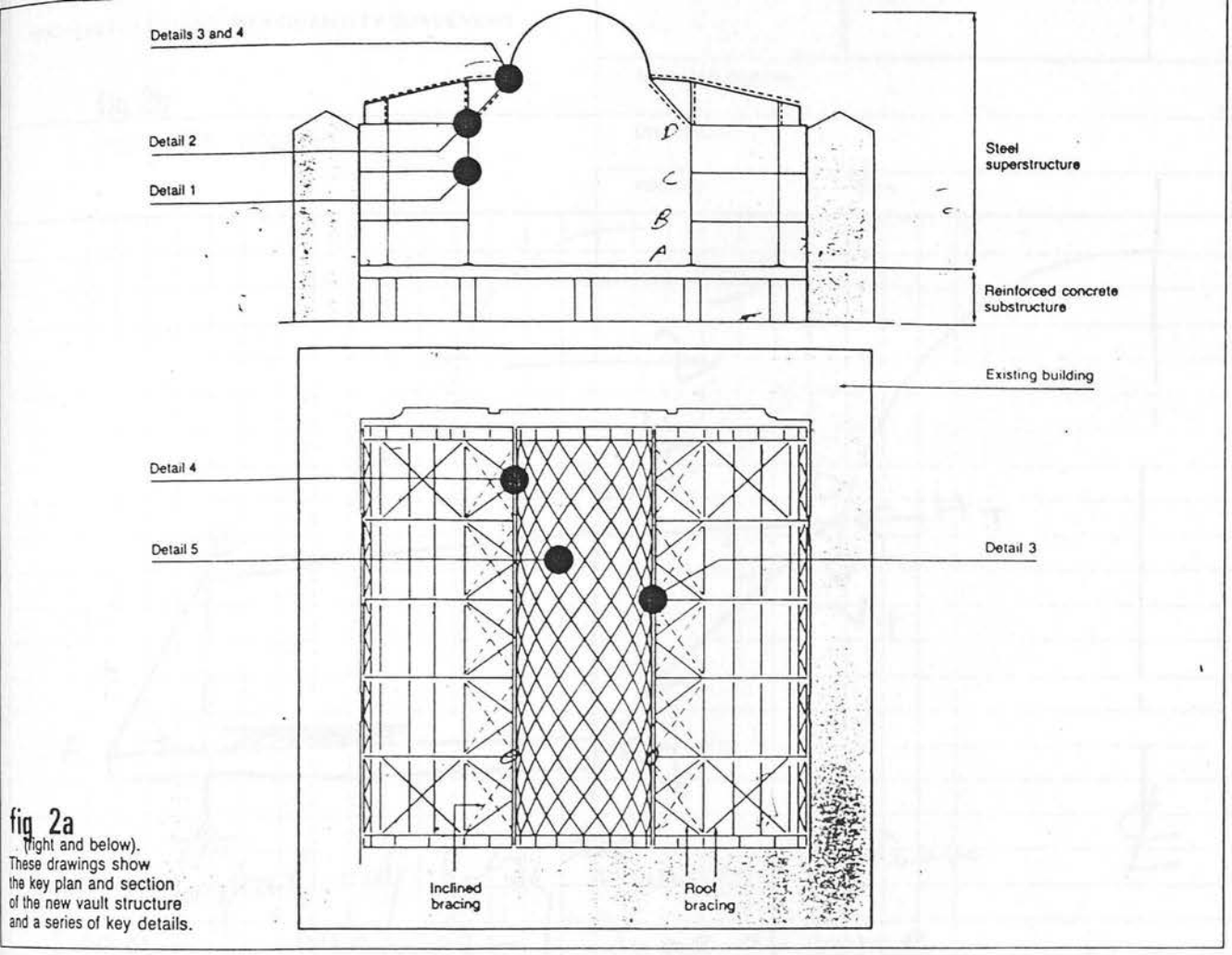
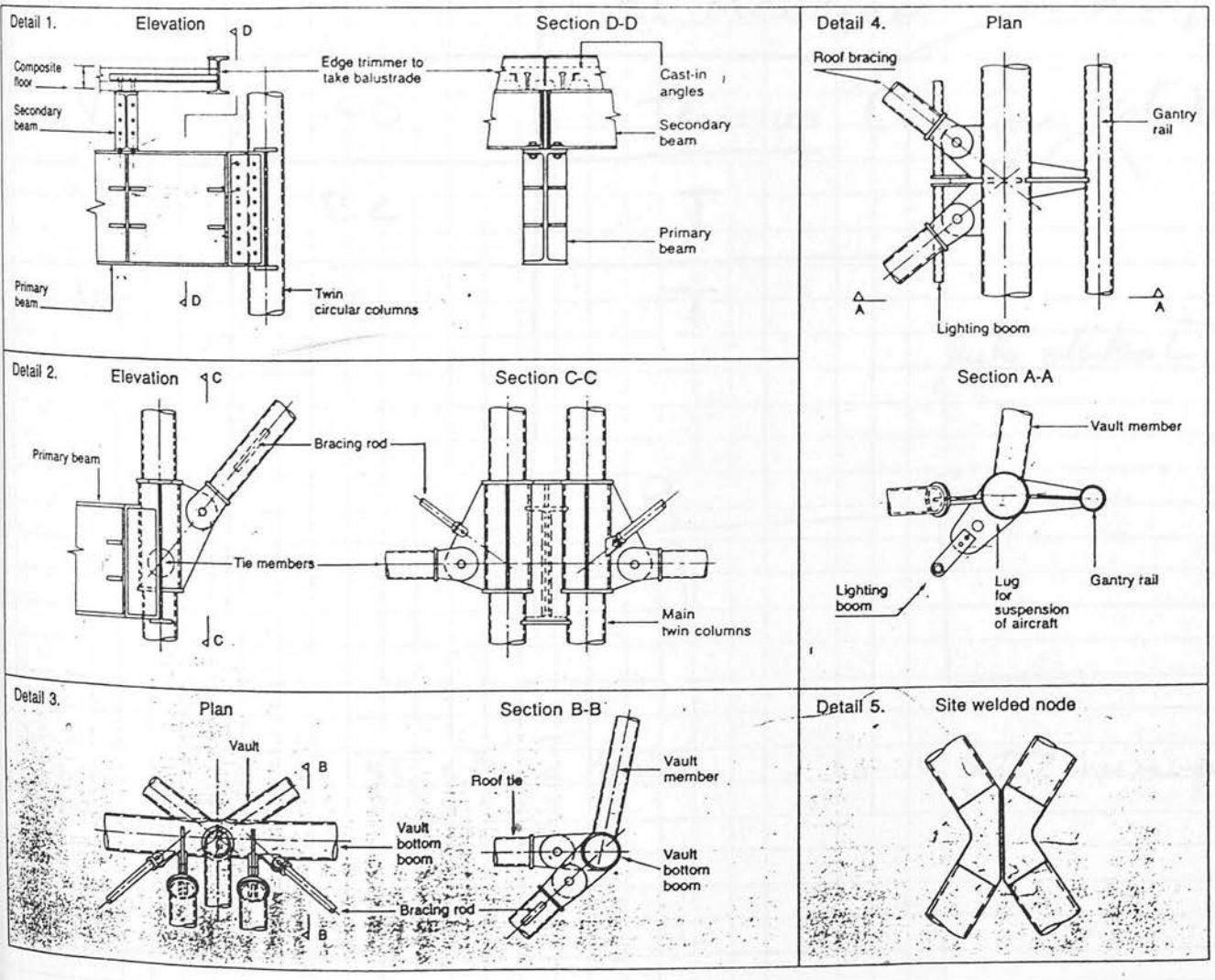


fig 2a
(right and below).
These drawings show the key plan and section of the new vault structure and a series of key details.



ARUP ASSOCIATES

ARCHITECTS + ENGINEERS + QUANTITY SURVEYORS

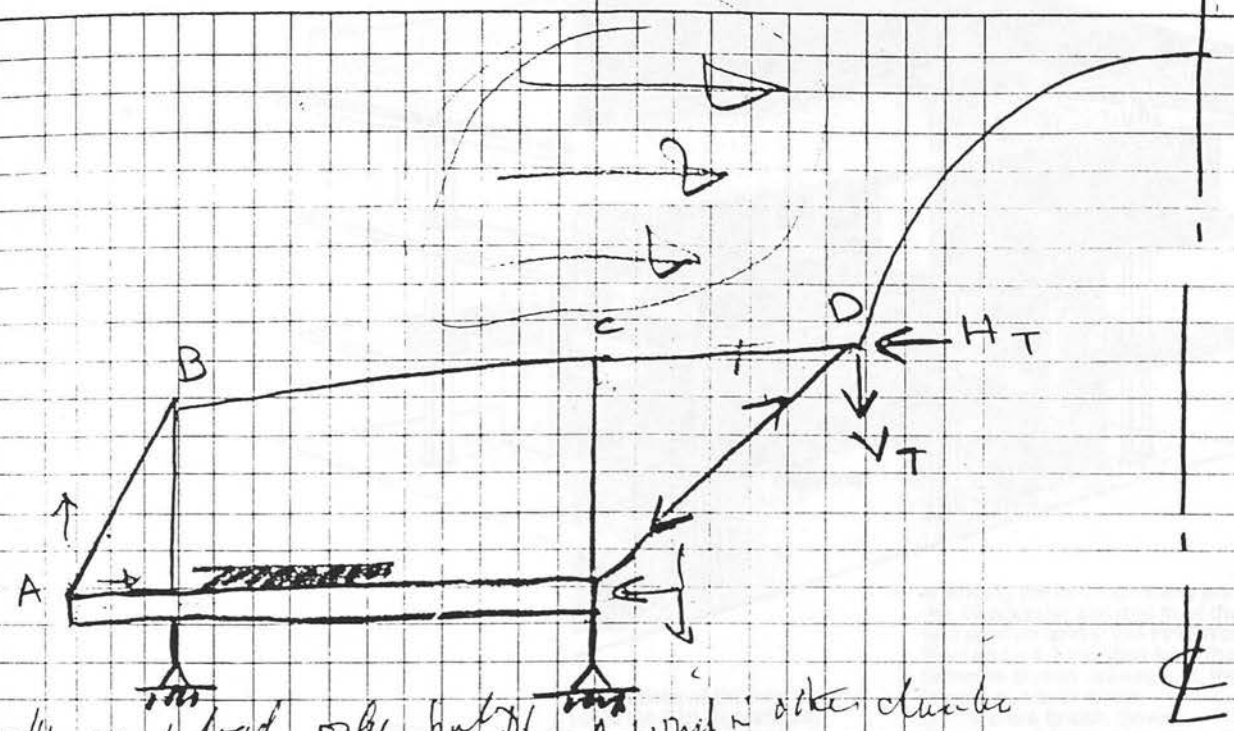
fig 2b

ENGINEER'S CONCEPT

Member/Location

Drg. Ref.

Made By Date



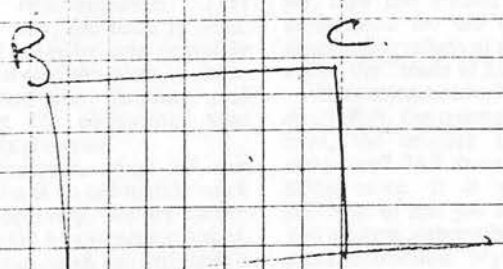
under wind load only but BC is within other chamber

Load	Member	Type of Force in Member
V_T	CD	Tension (T)
V_T	BC	T
V_T	AB	T

must be comp.

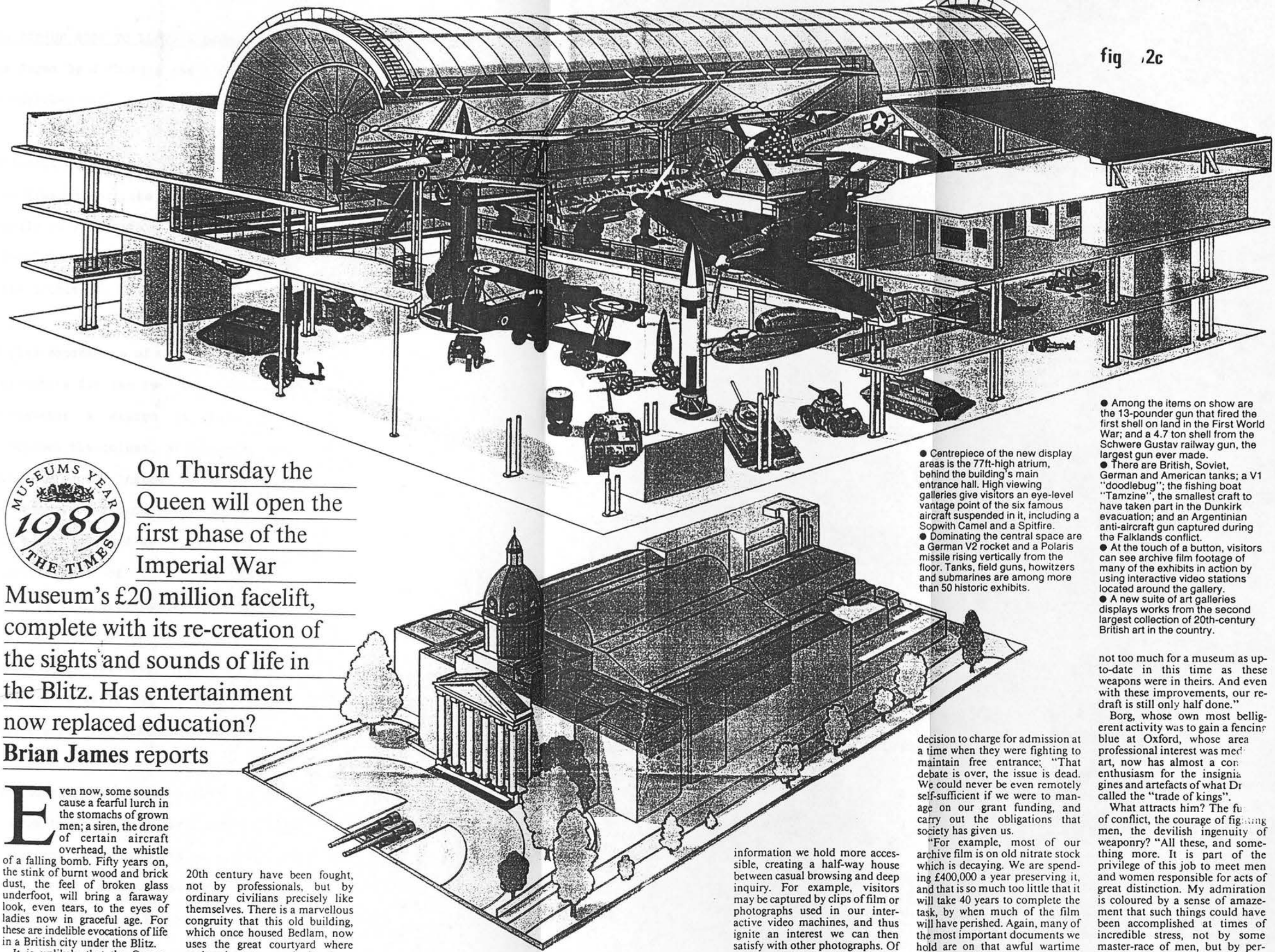
From vert. V of ⌒

further elaboration



As BC acts a portal ∴ so the portal has an effect

fig 2c



● Among the items on show are the 13-pounder gun that fired the first shell on land in the First World War; and a 4.7 ton shell from the Schwere Gustav railway gun, the largest gun ever made.
● There are British, Soviet, German and American tanks; a V1 "doodlebug"; the fishing boat "Tamzine", the smallest craft to have taken part in the Dunkirk evacuation; and an Argentinian anti-aircraft gun captured during the Falklands conflict.
● At the touch of a button, visitors can see archive film footage of many of the exhibits in action by using interactive video stations located around the gallery.
● A new suite of art galleries displays works from the second largest collection of 20th-century British art in the country.

● Centrepiece of the new display areas is the 77ft-high atrium, behind the building's main entrance hall. High viewing galleries give visitors an eye-level vantage point of the six famous aircraft suspended in it, including a Sopwith Camel and a Spitfire.
● Dominating the central space are a German V2 rocket and a Polaris missile rising vertically from the floor. Tanks, field guns, howitzers and submarines are among more than 50 historic exhibits.

MUSEUMS YEAR 1989 THE TIMES

On Thursday the Queen will open the first phase of the Imperial War Museum's £20 million facelift, complete with its re-creation of the sights and sounds of life in the Blitz. Has entertainment now replaced education?

Brian James reports

Even now, some sounds cause a fearful lurch in the stomachs of grown men; a siren, the drone of certain aircraft overhead, the whistle of a falling bomb. Fifty years on, the stink of burnt wood and brick dust, the feel of broken glass underfoot, will bring a faraway look, even tears, to the eyes of ladies now in graceful age. For these are indelible evocations of life in a British city under the Blitz.

It is unlikely that the Queen, exposed to a barrage of such triggers of the senses next week, will share the impulse of millions of her subjects to drop to the floor and scuttle beneath the nearest table. For hers was a slightly different war, as a schoolgirl in the relatively safer Windsor before

20th century have been fought, not by professionals, but by ordinary civilians precisely like themselves. There is a marvellous congruity that this old building, which once housed Bedlam, now uses the great courtyard where asylum inmates exercised to demonstrate the greatest lunacies of all.

The centrepiece of the new design is that courtyard, now a soaring atrium filled with the fascinating "toys" of war. One *Boys' Own* fantasy chariot, the Spitfire, hanging above our heads.

fill of these spectacular machines, we want them to start down the ramp . . . there to find the cost of war – 100 million dead in this century's conflicts, and 11 more years to go."

That ramp is cunningly done. Large, brave photographs show

familiar jumble of exhibits of the traditional museum, items of notable interest – from the royal grandmother's ration book, to the document that Hitler signed which ordered the invasion of Poland and began the Second World War; from the famous de-

information we hold more accessible, creating a half-way house between casual browsing and deep inquiry. For example, visitors may be captured by clips of film or photographs used in our interactive video machines, and thus ignite an interest we can then satisfy with other photographs. Of which we hold six million."

The quiet Dr Borg clatters like a battle-tank over the issue that brought crossfire from curators of other national museums – the

decision to charge for admission at a time when they were fighting to maintain free entrance: "That debate is over, the issue is dead. We could never be even remotely self-sufficient if we were to manage on our grant funding, and carry out the obligations that society has given us.

"For example, most of our archive film is on old nitrate stock which is decaying. We are spending £400,000 a year preserving it, and that is so much too little that it will take 40 years to complete the task, by when much of the film will have perished. Again, many of the most important documents we hold are on that awful wartime paper, and preserving them is another costly headache.

"If we asked at the door what we cost to run, our admission charge would be £12.50. I believe a fifth is

not too much for a museum as up-to-date in this time as these weapons were in theirs. And even with these improvements, our redraft is still only half done."

Borg, whose own most belligerent activity was to gain a fencing blue at Oxford, whose area professional interest was med art, now has almost a con enthusiasm for the insignia gines and artefacts of what Dr called the "trade of kings".

What attracts him? The fu of conflict, the courage of fighting men, the devilish ingenuity of weaponry? "All these, and something more. It is part of the privilege of this job to meet men and women responsible for acts of great distinction. My admiration is coloured by a sense of amazement that such things could have been accomplished at times of incredible stress, not by some master-race of men, but by perfectly ordinary people. If we can build a museum that conveys some understanding of this . . . this *awe* . . . we shall have succeeded."

CASE STUDY 8: ATRIUM ROOF TO LLOYD'S BUILDING

(Extract from Paper by J Thomson and P Rice 'The Structural Engineer', Vol 64A/No 10/October 86)

The upper galleries around the atrium were glazed, and this glazing had to be continuous part the roof and become the atrium glazing. There was really no alternative to this since the roof occurred at a number of levels because of the cutbacks. It was also preferred visually by the architect.

As part of this expression of continuous glazing in the atrium the supporting structure for the roof had to be outside. This made it possible to develop a design in which there was a consistent relationship between the column, glazing and supported structure, both above and below the roof. Instead of treating the steelwork above the roof as a new system, independent of that below, the concrete columns in the atrium were continued above roof level and used to support the steelwork. Derivatives of the floor brackets were used to pass through the glazing and carry the steel (Figure 5).

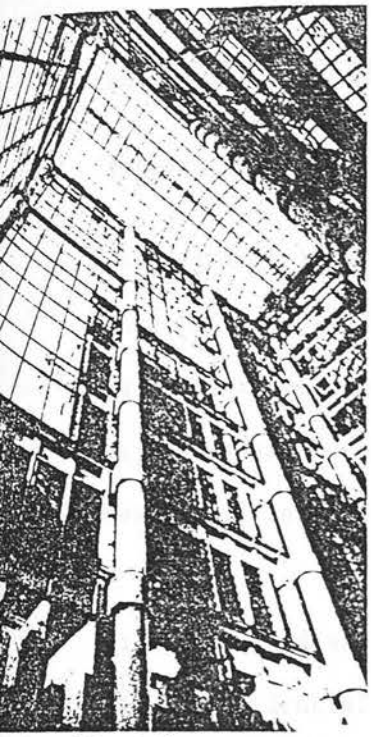
The structure had to be visually light from the inside. A space frame was considered for a while but its homogeneous appearance was not appropriate and a more hierarchical solution was devised.

A series of horizontal trusses run around the atrium, one level above each roof. From these trusses are suspended vertical trusses which support the glazing. The cutbacks are in increments of three levels which means that either one or three levels of glazing are supported, depending on the location. The uppermost trusses carry the barrel vault.

The horizontal trusses are 3-dimensional to carry both horizontal and vertical load, while the suspended structure is a series of planar trusses at 1.8 m centres. These are laced together by rails bolted to the boom adjacent to the glass which gives stability to the outer boom when this is in compression. The barrel vault is a three-pinned arch on the same principle (Figures 3 and 6).

The planar trusses are suspended with pin joints while a slotted bottom pin joint allows temperature movement and makes the load path determinate. The horizontal trusses also have slotted end-connections for temperature movement.

The design of the atrium roof calls for a large number of identical pinned connections between trusses. This detail as first conceived was quite labour intensive, consisting of a steel plate welded to the end of a tubular boom and rebated for sealants which would water-proof the joint. With this degree of repetition and complexity a steel casting became economic and looked much better. Castings were then used to support the primary trusses from the concrete column brackets for the same reason.



Atrium roof from inside

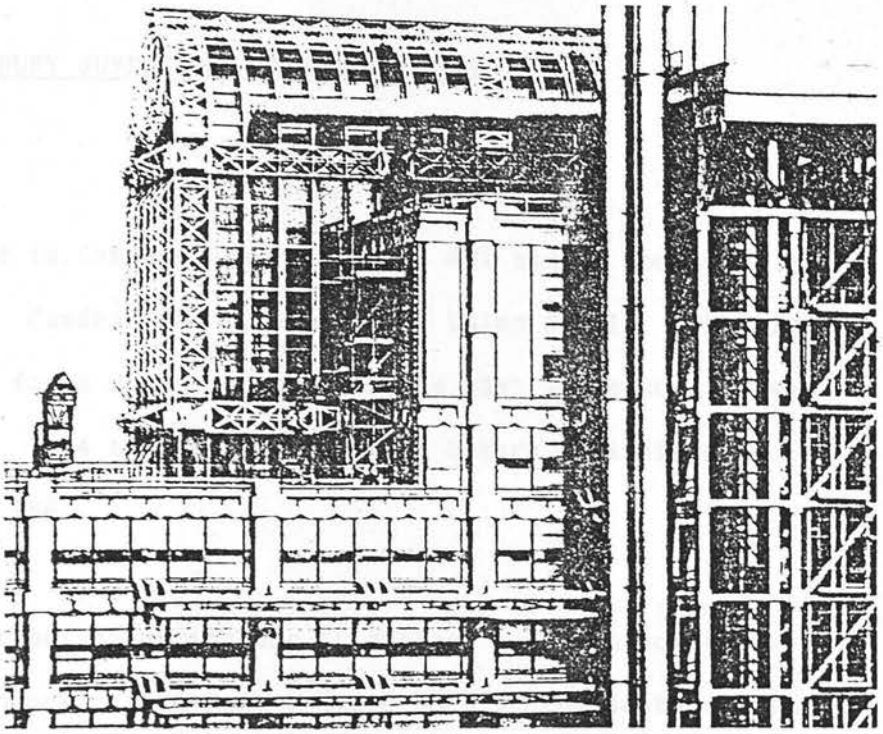


Fig 6. Atrium roof from east

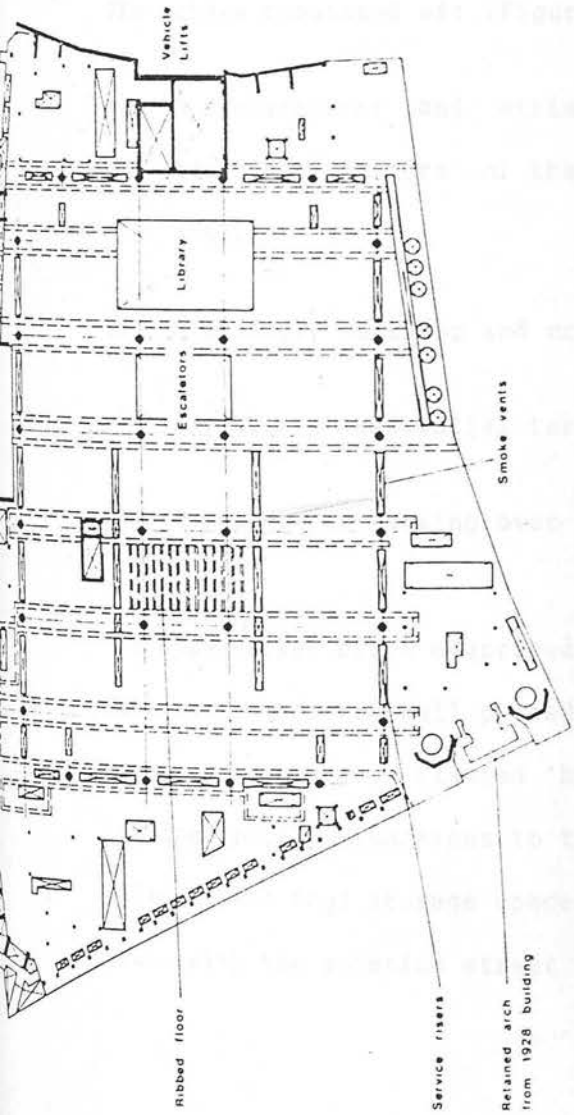


Fig 4. Lower ground-floor

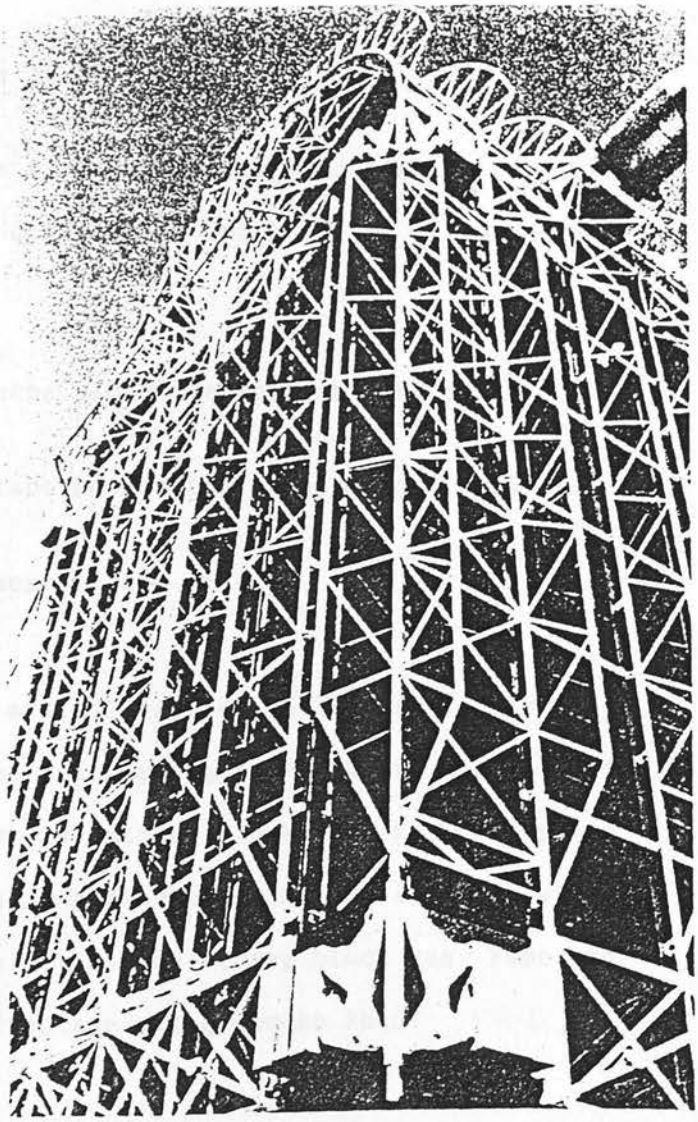


Fig 3 Atrium roof castings

CASE STUDY 9: SAINSBURY SUPERSTORE AT CAMDEN TOWN

Introduction

The site is located in Camden town, London, NW1 and is bounded by Kentish Town Road, Camden Road and the Grand Union Canal. The site had been developed for a number of uses in the past years and it was major bakery from 1924 to 1984. Existing bakery buildings were demolished in late 1986.

Significant adjacent buildings include St Michael's Church and a five-storey residential block called Barnes House both to the south of the site.

The scheme consisted of: (Figure 1A to 1B)

a) A supermarket and atrium fronting Camden Road (which is the principal feature of the development and the item of this case study).

b) A 2-storey workshop and creche alongside Kentish town road.

c) A 3-storey residential terrace beside the Grand Union Canal and

d) Basement car parking over most of the site.

The supermarket block comprised a single-storey vaulted roof (visually derived from Market Hall precedents according to N Sidor, Grimshaw's project architect, flanked by two-storey blocks on the periphery housing backup functions to the supermarket such as administrative offices and food storage spaces. The two-storey block was kept in scale with the existing street frontage along Camden Road.

Constraints

The main priority of J Sainsbury was to maximise the floor space within the given site by keeping the main supermarket floor area free from structural encumbrance. In this way maximum retail value could be ensured for the given space as structural obstructions to goods displayed compromise their sales potential and Sainsbury would be obliged to charge less on rent.

This meant that preparation areas, bulkstock, plant rooms and staff cafeteria were to be housed one level above the store level, in peripheral blocks. The scale of superstore was to harmonise with that of the surrounding Georgian buildings and as can be seen in Figure 3, the proportion of the ceiling height to the width of the retail floor showed almost minimum headroom to a space that would be occupied by several people. For this reason, the curved ceiling (in a market hall metaphor) provided the relief to an otherwise ill-proportioned space.

The planning and layout of foundations were constrained by:

a) The proximity of the Northern Line to Kentish Town Road which prevented the use of deep foundations adjacent to that boundary.

b) The proximity of St Michael's Church and its foundations from which the basement structure had to be kept away.

c) The Fleet Storm Relief sewer from which proposed foundations should be kept clear.

d) The existing foundations surrounding the oval sewer crossing the southern tip of the site, which was to be left intact.

e) Barnes House which prevented the installation of deep foundations adjacent to it.

The basement slab is designed as a ground bearing slab in conjunction with a system of ground beams and also to resist forces from upward water pressure. Consequently, a relatively close grid of columns is used to control the size of loads applied to the basement structure, which also permits the economic use of steel or reinforced concrete frames.

Development of the Structure

Main Roof Truss (Figure 1C)

The volume of the Market Hall below the roof covering is kept to a minimum by using a minimal clearance below the bottom chord for ceiling and lighting fittings. Open bays are provided towards the centre of the truss to allow free passage of larger services. Apart from centre bays, which are designed without bracing members in Vierendeel configuration, the truss is a conventional curved simple span supporting a lightweight roof.

Junction between Girder and Roof Truss

The junction between the cantilever girder and the roof truss was considered in a number of ways. As illustrated in the sketches above, option X was selected as it met the following architectural constraints:

a) The scale of the massive roof was to be kept low in height.

b) The creation of an interesting transition space provided visual relief to the main elevation (See sketch option X).

c) A structural joint which allowed for movement and ease of erection.

d) A satisfactory expression of the separation between roof and support elements.

Cantilever Girder

The cantilever girder is shaped to reflect the heavy bending and shear loads and takes the form of a tapered steel plate girder which gives the required strength and stiffness. It puts the main stanchion in compression and the tie-down rods in tension (Figure 2).

Main Stanchion

The main stanchion is designed as a concrete filled steel tube which has been chosen to provide a neat and strong junction bettween it and the cantilever girder.

Tie-down Rods

The tie-down rods are arranged in groups of four and are made of high strength vandal-proof steel.

Anchorage

Reinforced concrete piling with a safety factor against uplift is used to anchor the tie-down rods through a pile cap.

The Emergency Truss

This is a safety measure against structural failure should the tie-down rods be severed through acts of vandalism. Ductile steel rods are used in this emergency truss to ensure adequate resistance to out-of-equilibrium forces caused by a destabilised girder illustrated in Figure 4.

The Atrium

The lightweight transparent roof was an appropriate solution to the unheated entrance atrium. The structural frame is detailed and articulated to follow the curved line of the adjacent supermarket roof. Transverse trusses resist dead and snow loads whilst an orthogonal system of tie-rods resist the uplift caused by wind loading (Figure 5).

The Preparation Area

This structure is designed using shallow steel plate girders which tie into the supermarket grid. The depth of these girders is determined by clear height requirements in the unloading bay. The girders are designed compositely with the reinforced concrete slab which is cantilevered to facilitate movement in a very tight area. The preparation area is roofed by a lightweight steel roof.

Store Tie-down Foundations

Both diaphragm walls and contiguous pile walls were considered during the early design stages as means of forming the basement perimeter and as a suitable means of anchoring the cantilevered girders from uplift loads. These options were abandoned when it became possible to reduce

the size of the basement by moving the perimeter walls inward. This enabled the use of more economical bored cast in-situ piling with heavily reinforced pile caps. The pile caps provide an increased lever arm between zones of tension and compression resistance (Figure 6).

Structural Stability

Each part of the store is designed for normal stability criteria as described.

The Store, Atrium and Preparation Areas

The rear block and preparation area are tied together and braced by three shear walls located in the plan on Figure 7. The front block is unbraced in the direction of the clear span and stability is achieved by using the stiffness of the tapered girder and main stanchion frame junction (likened to portal frame rigidity). The block is braced by means of shear walls in the longitudinal direction.

Settlement

Settlement and ground movement is controlled by designing pad footings with allowable bearing pressures. The loads from the residential block are relatively light and although supported on three different foundation systems (See Engineer's Report KL & P) no significant differential settlement is anticipated.

Special measures were taken to minimise vertical movement in the main tie-down anchorages. This is necessary because the tapered girder magnifies these movements at the main roof connections. Stringent settlement and long term creep limits were imposed on the pile design

and the link between the piles and tie-down rods is post-tensioned to control elastic extension under load (Figure 8).

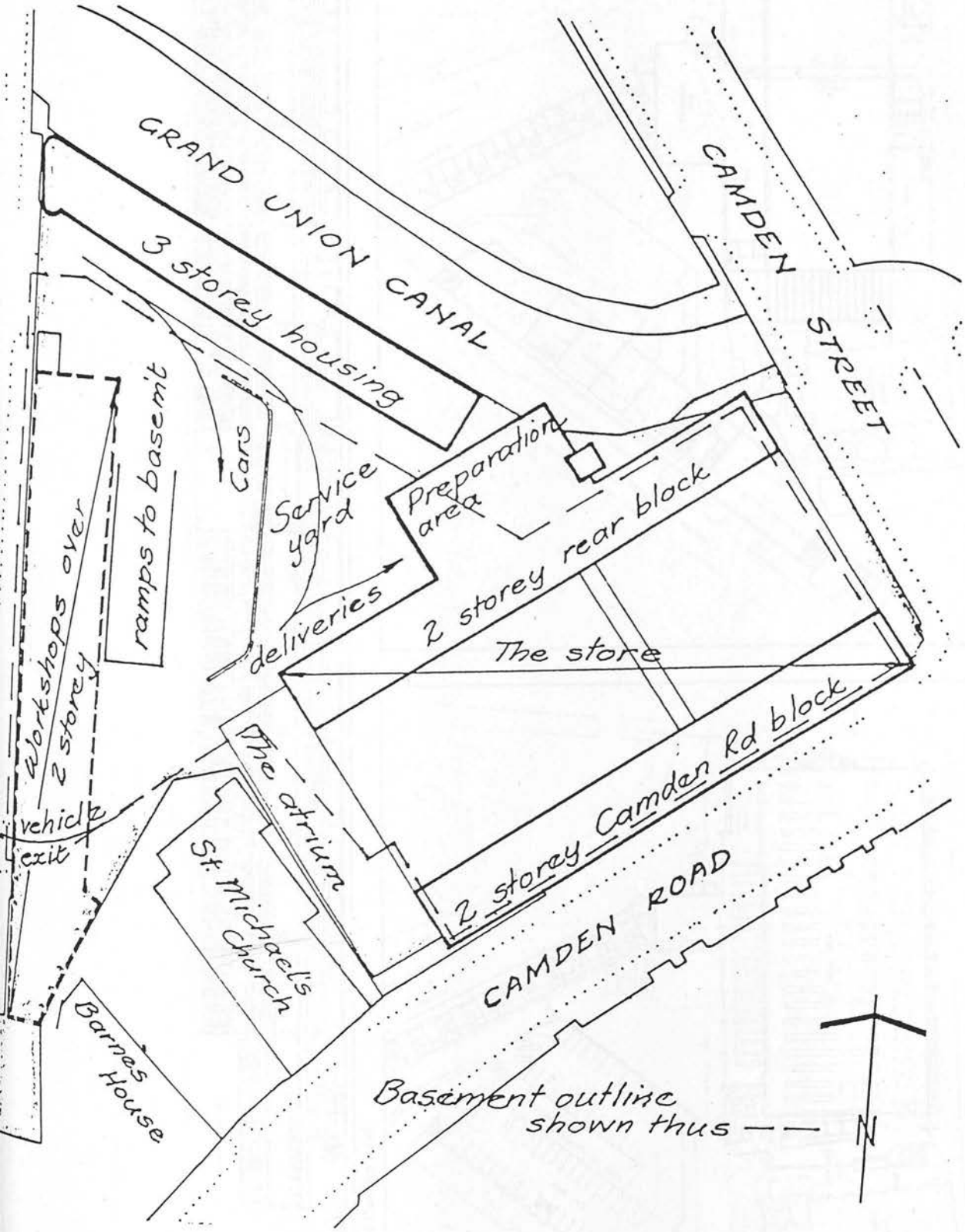
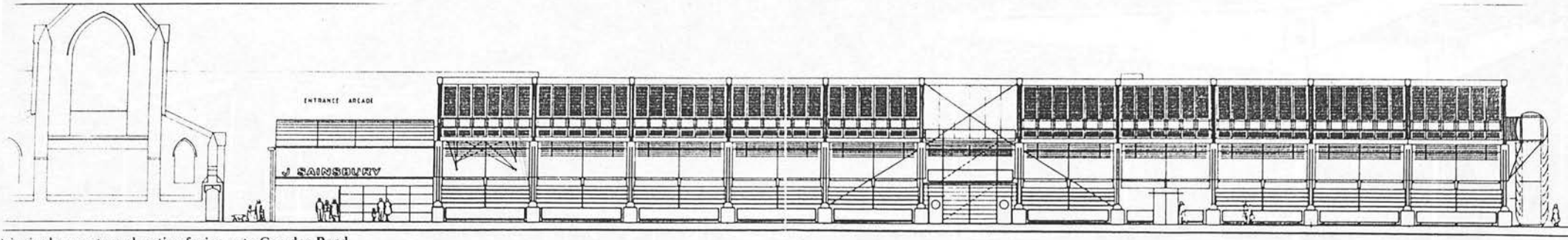
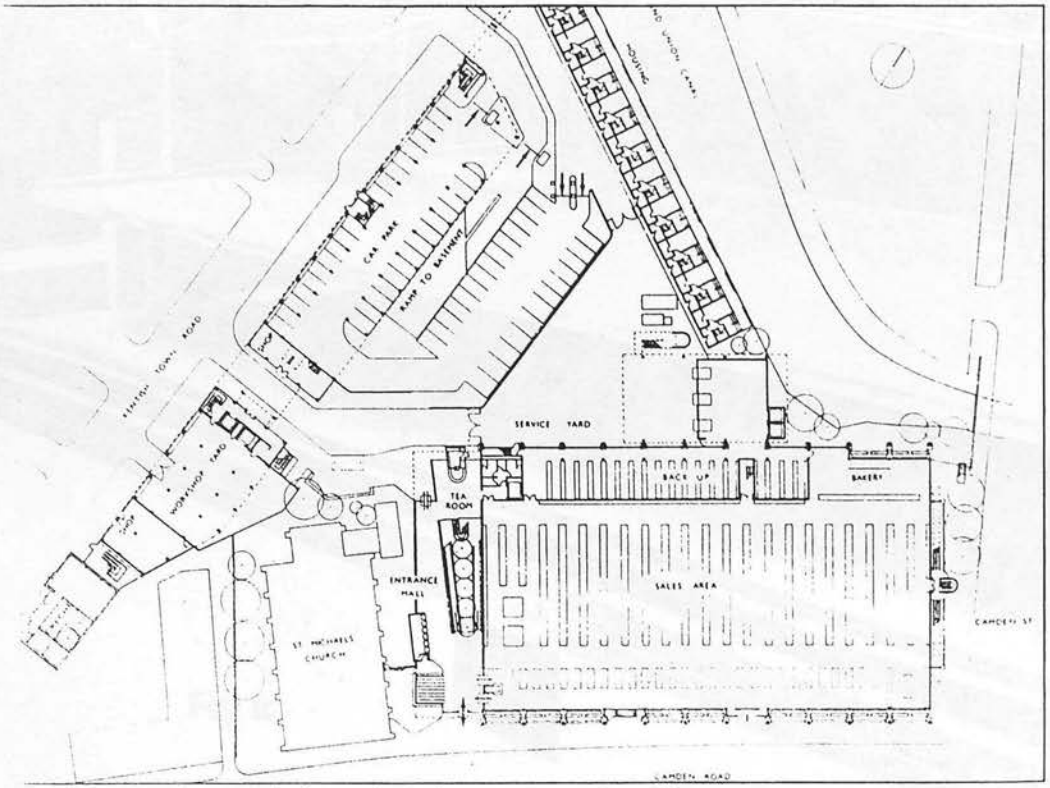

GRAND UNION CANAL
3 storey housing
CAMDEN STREET
Cars
ramps to basemt
Service yard
Preparation area
2 storey rear block
deliveries
The store
workshops over
2 storey
vehicle exit
The atrium
St. Michael's Church
2 storey Camden Rd block
CAMDEN ROAD
Barnes House
Basement outline shown thus
N

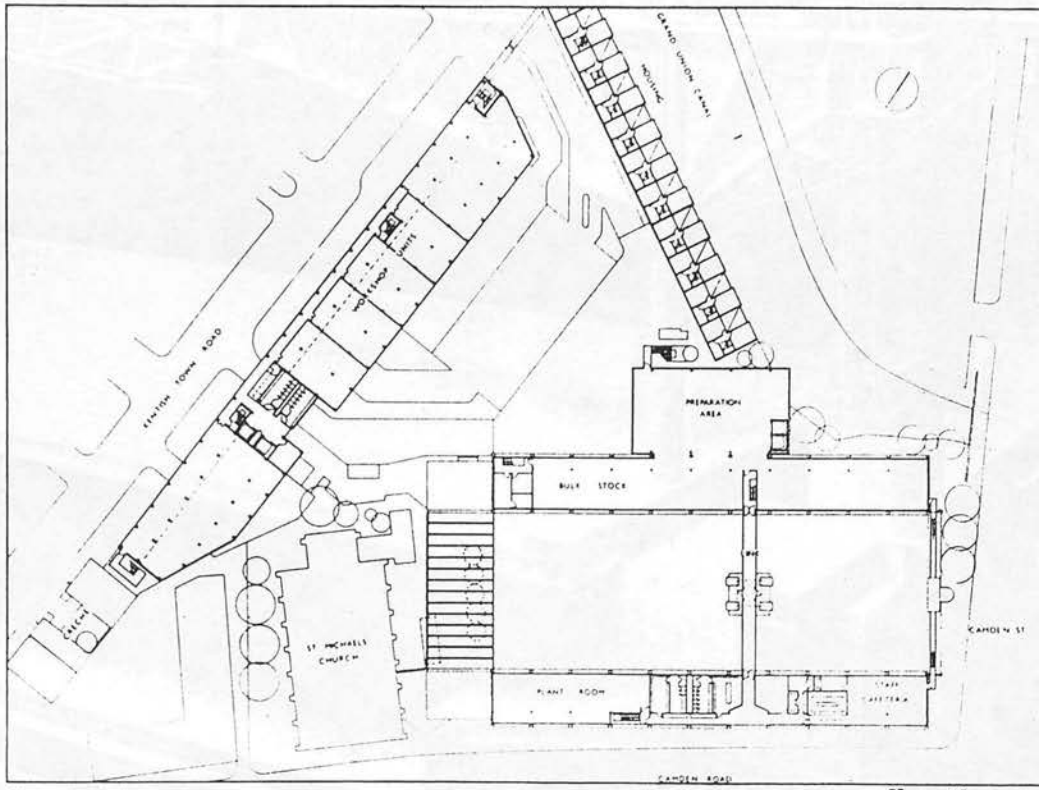
Figure 1a



'rincipal superstore elevation facing onto Camden Road.



'ound-floor plan; a basement level across the site incorporates extensive parking.



Upper-level plan.

fig 1b

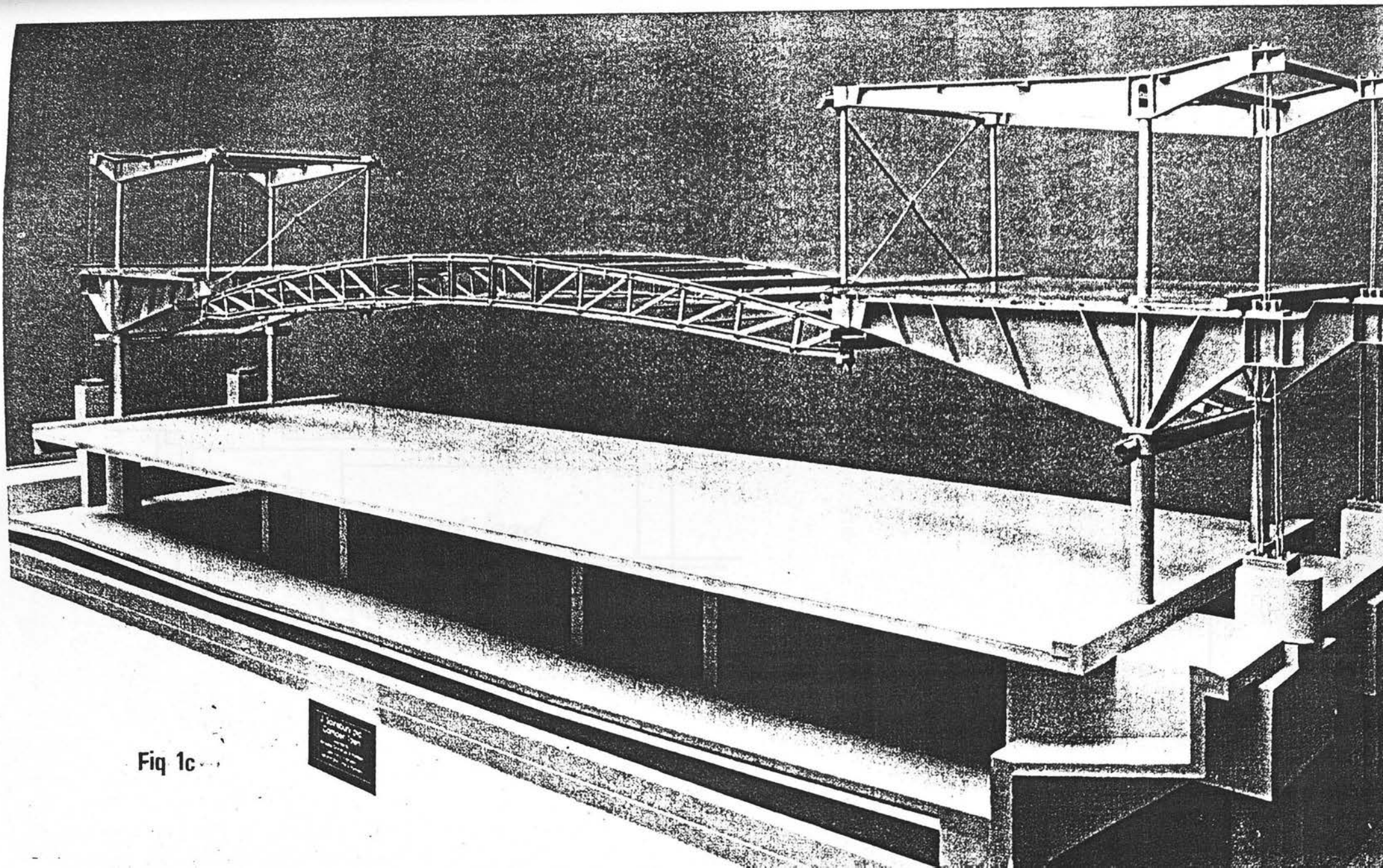


Fig 1c

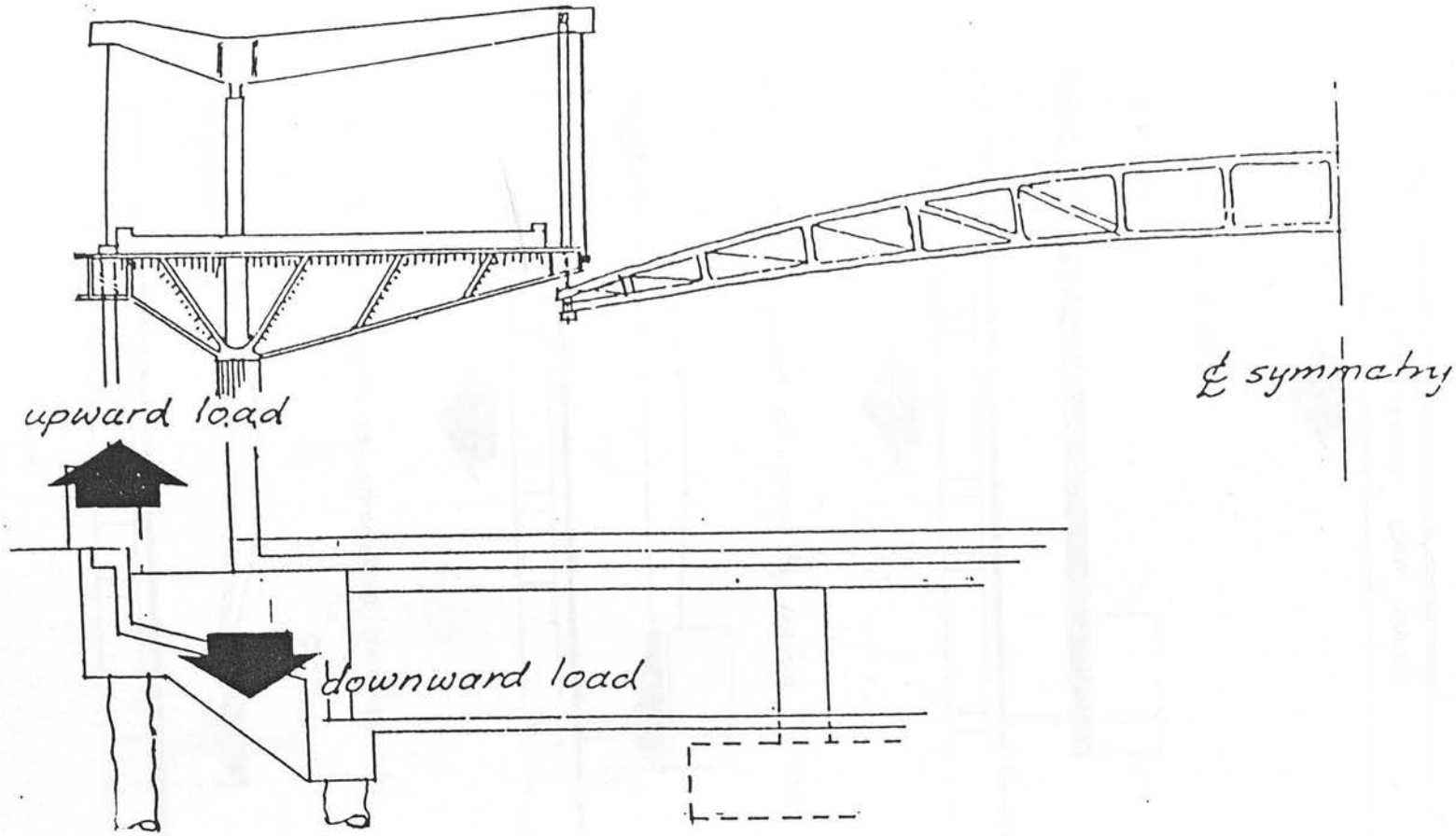
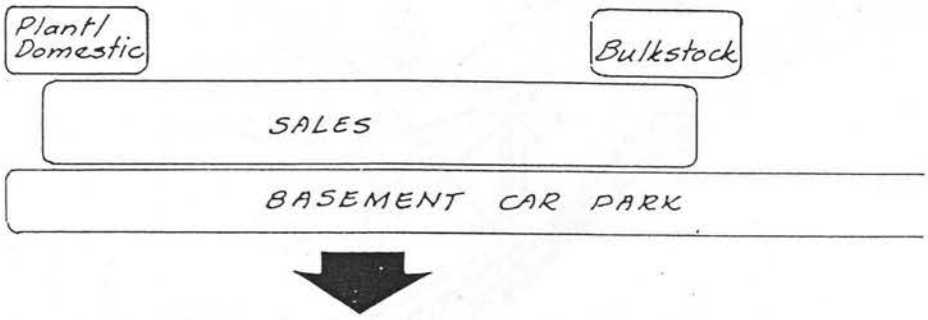
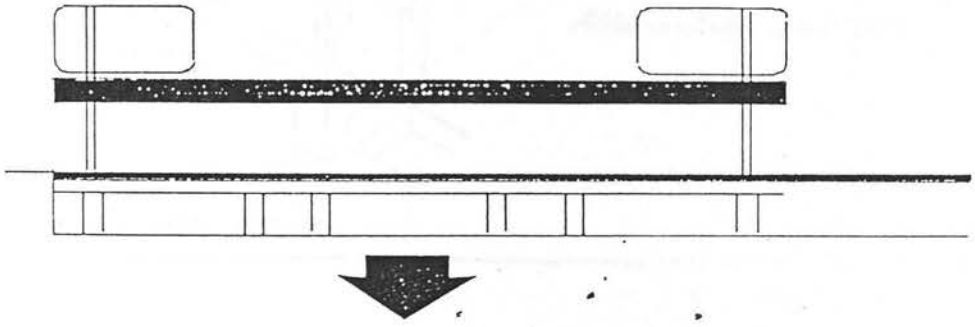


Figure 2.

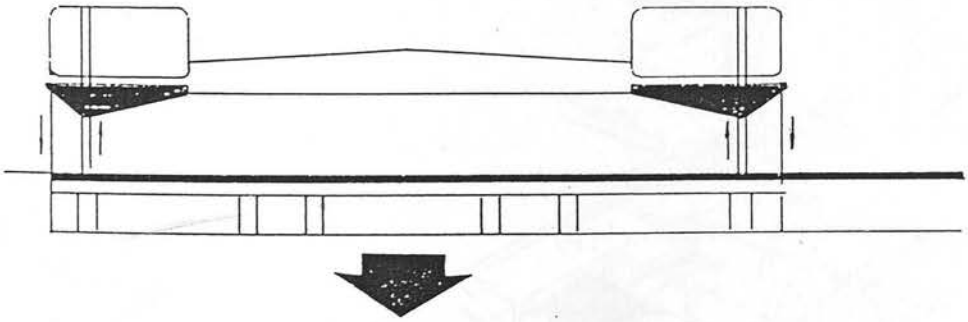
Position 1st floor accommodation close to perimeters



Introduce clear span



Lighten central part of span by using tie-rod backstays



Curve ceiling & express structure externally.

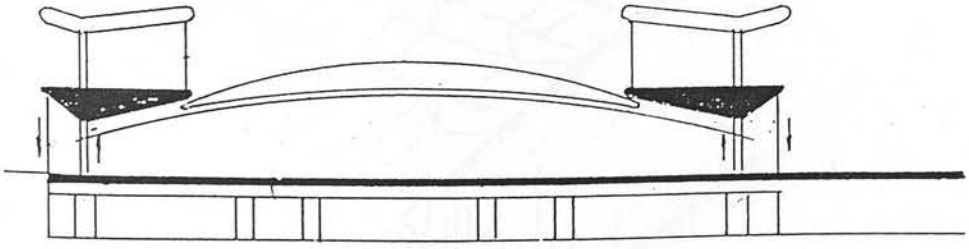


Figure 3.

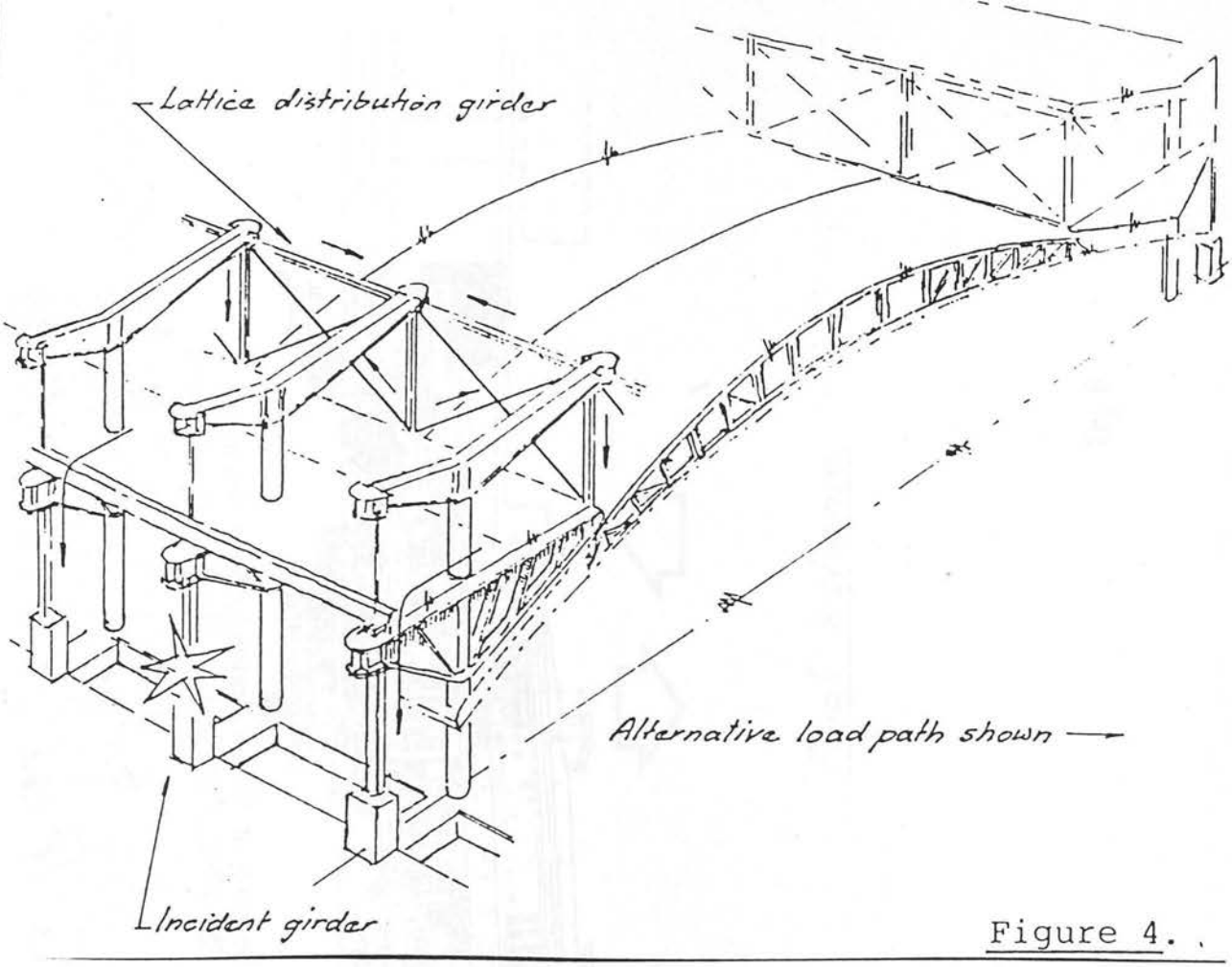


Figure 4.

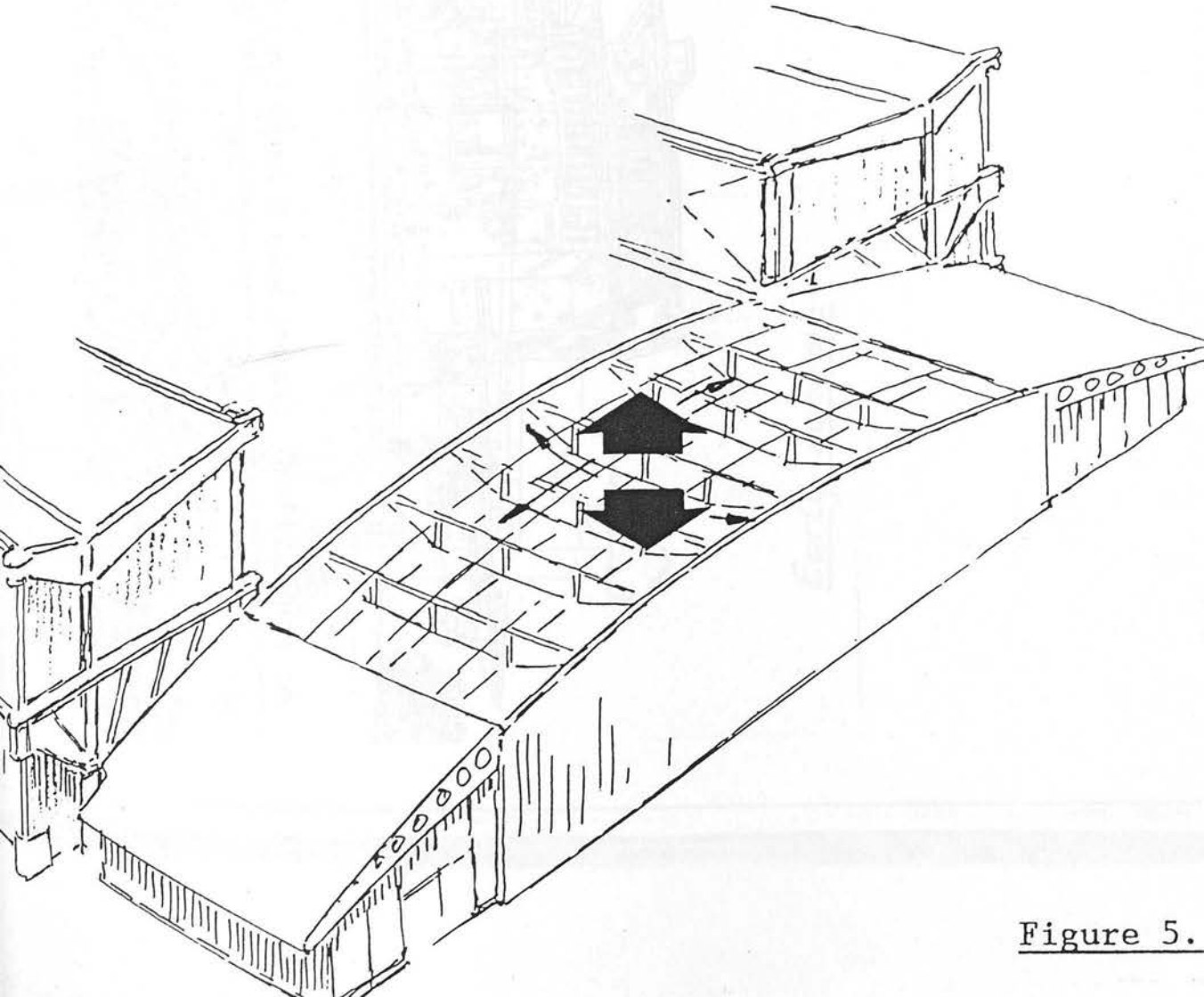
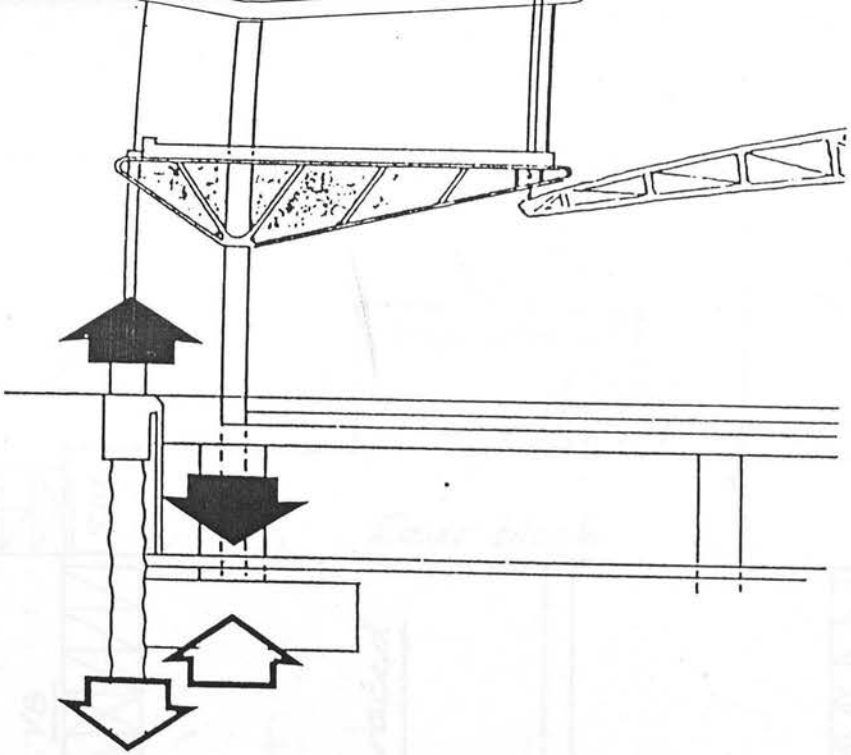
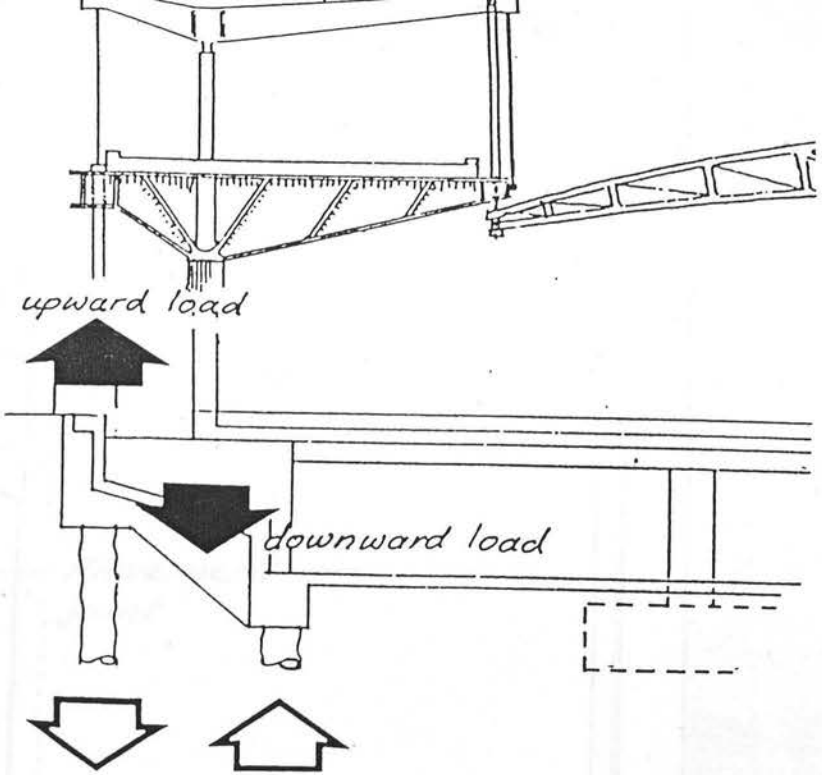


Figure 5.

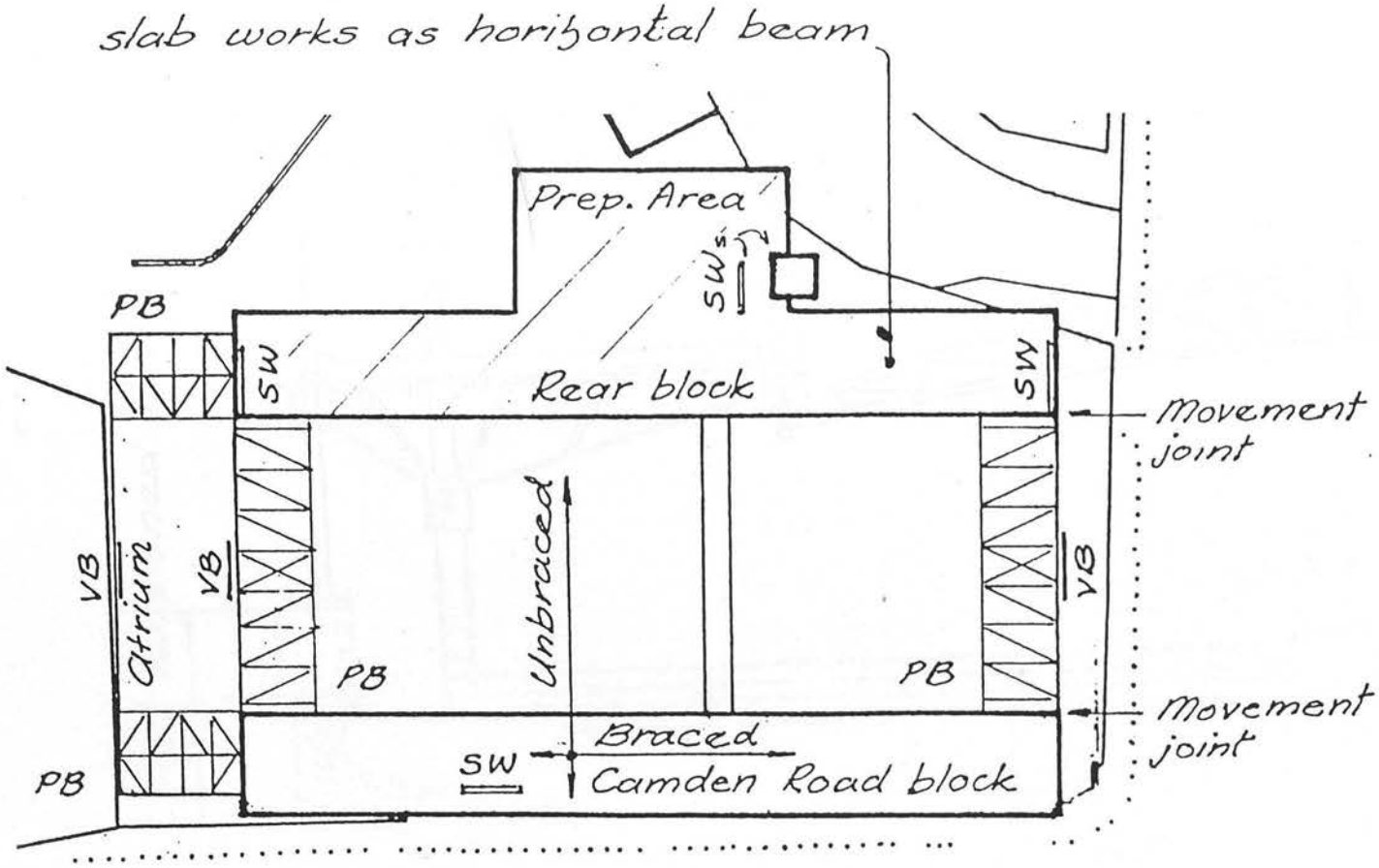


Early scheme



Final scheme

fig 6



SW = shear wall

VB = vertical bracing

PB = plan bracing

Figure 7.

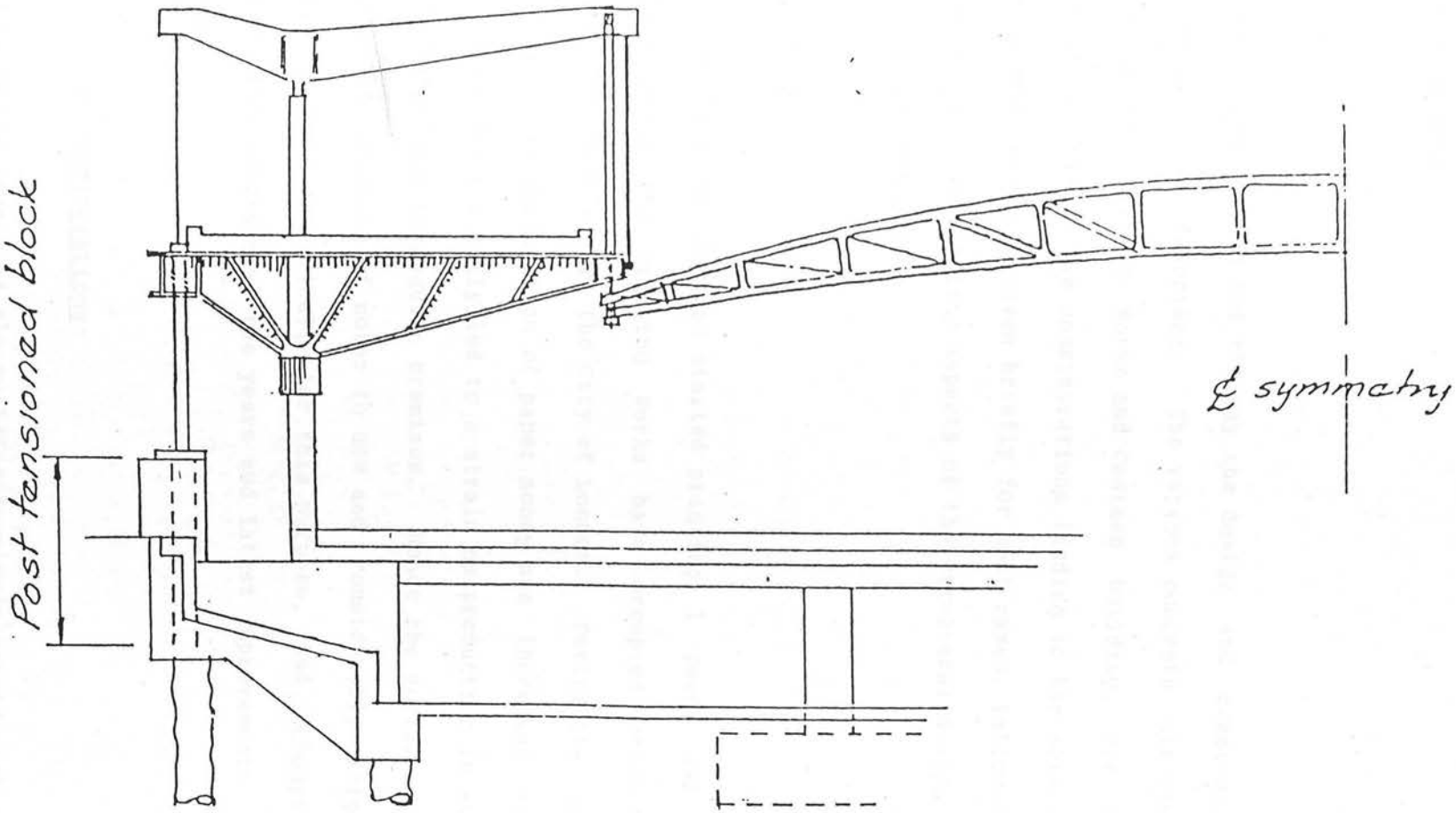


Figure 8

CASE STUDY 10: DESIGN AND CONSTRUCTION OF THE PRINTING WORKS AT DEBDEN

(Architects, Easton and Robertson, Structural Consulting Engineers, Ove Arup and Partners)

1.0 Synopsis

The general background and the way the design and construction was organised are described. The various concrete structures comprising the Printing Works and Canteen building, are then dealt with in turn. The considerations leading to the selection of structural forms are given briefly for most cases, followed by some of the more interesting aspects of the structural design and constructional methods.

2.0 Introduction

Since the Bank of England started printing 1 pound and 10 shillings notes the Printing Works have occupied converted premises, St Luke's, in the City of London. During the past twenty years the circulation of paper money has increased many times; this has naturally led to a strain on production in what are now cramped and inadequate premises. Hence the decision to transfer the printing of notes to new and considerably larger premises designed specifically for this purpose, and embodying the experience gained over the years and latest improvements in technique.

3.0 General Design Considerations

In arranging the layout of the building four main considerations were kept in mind.

1) Clear floor spaces and freedom from overhead services.

2) Good production flow.

3) Control of all processes with the minimum of expense and of security supervision.

4) Good working conditions.

The Main Hall, about 800 feet long, forms the basis of the plan and it was decided to place this parallel to the railway (the building is capable of extension towards the railway).

The fall across the site was accommodated by making the ground floor level at the high side of the site the first floor level at the low side; these floor levels are referred to as upper ground floor and lower ground floor. The site between the main building and the railway was levelled off, the railway being protected by a retaining wall varying from nothing at the east end of the site to about 11 feet at the west.

The complete buildings are shown in the aerial view in Figure 1, and comprise the Printing Works and the Canteen.

The Printing Works consist of the Main Hall which occupies the whole of the north side, three- and five-storey office buildings and laboratories on the south and east side, a small hall on the south-west of the Main Hall, and the inner core with general printing area and storerooms. The boiler house and turbine generating room are situated underground below the office and laboratory buildings at the east end. Apart from the office

buildings the factory area is planned on two levels (upper and lower ground floor). On the upper ground floor is the printing area; the lower ground floor accommodates stores, cloakrooms, lavatories, heating and ventilation plant rooms and, in the area at the north of the Main Hall, a crawl space for services and ventilation ducts. At the west of the site and separated from the main building (connected by a tunnel) is the Canteen and Recreation Hall building. This block consists of the Kitchen, Dining Hall and Recreation Hall. Each of these areas is covered by different types of shell roof.

4.0 Practical Considerations for the Choice of Structure

Factory services generally are carried in horizontal ducts in reinforced concrete below the lower ground floor slab; these ducts start from the boiler house and spread over the whole factory. Under the Main Hall generally the services are raised to the underside of the upper ground floor. These ducts carry electric cables, heating pipies, water mains aand sewage pipes. Vertical services are taken in ducts, often formed by using the space between twin columns or twin arches.

The main lines of the structural design were closely adapted to the functional and architectural requirements. Concrete was chosen as the structural material throughout, partly for reasons of economy and obvious suitability and also - in the case of the long span roofs, where structural steel might have been competitive - because all internal surfaces should be as free as possible from dirt-collecting projections and excrescences, the condition of which strongly favoured the use of concrete shells.

The considerations which led to the special forms used in the various parts of the building will be dealt with later.

After the structural design was decided in outline, there still remained the question of how the structure was going to be built. Normally this is a question which is left to the Contractor - and this may be reasonable enough in the case of small jobs of orthodox construction. But this job was neither small nor orthodox, and it posed problems for the Contractor which could be solved in several ways, and the solution of which would greatly affect the details of design.

It was obvious that full use would have to be made of the repetitive features of the design by the construction of large, mechanically movable units of formwork and scaffolding, or by the adoption of pre-casting techniques, or both, and this for the sake of speedy construction as well as of economy, because the programme was a very tight one.

The Contractor was anxious to use pre-casting techniques as much as possible, or at least everywhere where it would help to ensure accuracy in the positioning of pre-stressing cables and fixings for mechanical equipment, to obtain good finishes on exposed work, or to save time and money. They decided to cover the site with two 5-ton derricks, and this determined the size of pre-cast in one piece, whereas this was not possible with the larger shells for the Main Hall, where for this and other reasons another technique was adopted. As the cranes could also be used for concreting in-situ, it was possible to mix pre-cast with in-situ construction as desirable throughout the job, and this was

in fact done. Pre-casting was also employed for many of the smaller items such as lamp posts, cills, gutter-sections, etc.

5.0 Influences on the Choice of Structure for the Upper Ground Floor

The Upper Ground floor for nearly the entire Printing Works was of flat slab construction witth or without column caps according to the slab span and thickness. This form of construction was chosen mainly because of the multitude of pipes, conduits and small ducts which were to run at ceiling level in the Lower Ground Floor. The presence of beams would have added considerably to the cost of all these services as well as leading inevitably to a very untidy appearance. This Upper Ground Floor had to be designed for rather heavy plant. Some types of presses deliver a sort of hammer blow at each revolution and experience had shown that this type worked loose in the course of time unless held down very firmly indeed. The presence of this very strong floor had a great influence on the constructional methods adopted for the superstructure because it provided a suitable platform for large travelling cranes to handle pre-cast units weighting up to five tons.

6.0 Influences on the Structure of the Main Production Hall

The Main Production Hall is 125 feet wide in the clear and approximately 800 feet long. The length of the Hall lies as near to the east-west direction as the site would allow. The processes to be carried on in the Hall required good and uniform north light if natural light was to be used in the daytime. The aspect of the Hall lent itself to trnasverse penetration by lifht from the north (approximately) provided it was baffled at

suitable intervals to elimiante the uncomfortable glare that would have been present had very long stretches of glazing been visible at working level. To obtain the greatest intensity and evenness of natural illumination in a high hall the windows should start as low down as possible on the northern side and rise towards the centre of the Hall. The arrangement had to be such that the sides of the Hall should be hearly vertical so as not to obstruct working space or the Security Gallery which had to be provided all round at First fllor level. In addition a maximum clear height of 35 feet was decided upon to prevent the roof structure from being oppressive to workers in this large hall. A narrow band of south light through obscured glass was required along the southern side.

All these considerations led to spanning the Hall by arches. The asymmetrical shape, arising from the one-sided incidence of natural light and the steep sides made it clear that the arch form would be far from the structurally ideal of approximately parabolic. However with arches deep enough to act as baffles by containing the arcs of the shell roof spanning from arch to arch, it was anticipated that a relatively small amount of pre-stressing, having regard to the span and weight of the structure, would be sufficient to control the thrust line between the limits of the upper and lower core points (the middle third of a rectangular section). To save space it was desired to make the width of the arches as small as possible at floor level. The function of the roof structure had by this stage been sufficiently defined to make it possible to sketch in the inner and outer profiles of the arch. When it was considered, however, how such a sketch was to be "blown up" to full size to make an

accurate basis for setting out a smooth curve it seemed to be a good idea to replace it by some mathematical function which would enable ordinates to be calculated, and would ensure that each part of the curve was related to all the rest - a relationship that is not to be obtained by making up the curve from parts of circles of different radii.

7.0 Influences of Construction

In a hall of this length, with longitudinal north light shells, provision for intermediate roof drainage had to be made. It was a requirement that drain pipes, air conditioning trunking, lighting conduits, extract fans, and all servicing equipment of this kind should be out of sight. It was therefore decided to construct the arch ribs in pairs, with a 3 feet 6 inches gap between them. Although longer spans for the shells would have been possible, the most convenient spacing of pairs of arches, also taking into account their function as light baffles, was found to be 36 feet centre to centre. The thickness of each arch was made 9 inches, which was thought to be the practical minimum for two layers of pre-stressing ducts as well as mild steel binding rods to be accommodated in this thickness.

With these gaps, each bank of shells bounded by an arch at each end is an independent structure from upper ground floor level upwards, and no expansion joint problems arose. However, the security galleries, which were structurally trough shaped beams supported by the arches, were giving an expansion joint at every third bay of 36 feet.

The paths of the 12 wire, 0.2 inches pre-stressing cables are shown in Figure 6. As the rain water outlets through the arch from the roof gutters had to be by-passed by the cables, the two layers became inevitable.

The heavy reinforced concrete flab slab floor, already described, acted as the arch ties. The arch legs were notched back to 3 feet in width at floor level and by the provision of helixes of reinforcement in that locality and by locating the centroid of the pre-stressing cables on the centre line of the arch at floor level the arch was made structurally two-hinged. This device was not only necessary structurally to keep the arch legs as narrow as they are, but it also simplified the design. The tie force, which was the redundant in this calculation, was reduced by the pre-stressing and was different for each trial layout of the latter. The reason, of course, was that the cables were near the lower edge over much of the central part of the arch. The most critical condition during construction was immediately after pre-stressing, when the full creep of the concrete would not have taken place and before the 3 inches of Leca concrete for insulation had been placed over the shell roofs. The thrust line, at this stage, in the pre-cast part of the arch was just above the lower middle third. In the final condition, as can be seen from Figure 7, the thrust line rises well away from the lower middle third. It can also be seen that the arch will have a high load factor of safety because with increasing superimposed load the thrust line would rise above the upper middle third causing cracks to begin on the lower edge (where the pre-stressing wires are) and these cracks would close up upon removal of the load.

8.0 Influences of Structural Shape on Construction Method

In the twenty-two structural units of the roof of the Main Hall, approximately 50 per cent of the concrete is in the ribs and the remainder in the shells and valley beams. It was visualised that to construct this entirely in-situ would involve complicated shutters to the sides of the arch ribs and delays would inevitably occur due to the concreting of the arches being interrupted by that of the shells. The accurate location of the duct tubes would also be extremely difficult from their position at the bottom of shutters 10 feet deep and 9 inches apart. The method of construction adopted was to cast in-situ the arch haunches and to make the central part from pre-cast lengths each weighting five tons. The latter were cast on the flat on a concrete bed where the curve of the arch was accurately set out and identical repetition ensured. The duct tubes were also accurately located where they passed through the shutters forming the division of the units. Figure 4 is a view of the pre-casting beds. The duct tubes visible had not yet been blown up with water. The steel left projecting for bonding into the shells, which were cast in-situ, can be seen.

A travelling gantry was made for the erection of one complete structural bay consisting of a bank of shells bounded by an arch rib on each side.

9.0 Structural Considerations in Shaping Structure

The plotting of the positions of resultant thrust lines was a fruitful way of looking at the problem of pre-stressing an arch, a portal frame, or even a beam for that matter. A thrust line

for the homogeneous material (before the additional thrust due to pre-stressing was added), was that approximately parabolic shaped diagram denoted by DL + LL. It might be supposed from the large bending moments indicated by that diagram, that, just as in portal frames of steel and reinforced concrete, the distortion due to direct force and shear were of negligible influence compared with bending, in determining the redundant tie force.

However, the additional thrusts due to pre-stressing had the effect of moving the thrust line, under the most prevalent condition of dead load only, very nearly onto the centre line of the arch. Thus, in fact, the resultant bending moment influence coefficient was not large compared with that of direct force, and it was important to take into account elastic and plastic shortening of the arch in determining the redundant, which in turn determined the position of the thrust line.

If the arches had been thicker and much more pre-stressing had been used it would have been possible to eliminate the redundant reaction. The arches would then have become beams and the economy of a partial arching action would have been thrown away. that did not mean, however, that the problem of creep stress effects would have been removed. On the contrary, they would have been very important: the condition visualised was the arch with the legs resting on rollers. It would be equivalent to a tied arch without thrust when the inward movement of the rollers due to elastic and plastic direct strains in the arch was exactly cancelled out by an equal and opposite outward movement due to bending strains.

10.0 Discussions on Design Decisions for the Structure

Questions were raised by the audience to which this paper was presented. Most pertinent were criticisms on the approach to create an asymmetrical arch to accommodate standardised shell baffles instead of creating symmetrical arches to shell baffles which could have been designed to respond to the North Light requirements. The excessive structural depths of the beams supporting the roof of the General Printing Hall were also criticised as being visually bulky:

"It is evident that the chief criterion for the design of the roof structure for the Main and Small Halls was the provision of uniform north light. The span of the shells, the curve of the arches, their depth, were determined on this basis and therefore lacked the symmetry and slenderness which would have resulted in a cheaper, more beautiful and still less 'oppressive' roof structure. It has also complicated roof drainage problems. To provide uniform natural light is it not possible to have symmetrically positioned glazing in the plane of the roof but with different and varying refracting and diffusing media depending on their position, instead of distorting a roof structure to provide suitable natural light it will be more economical to develop and design special glazing materials to admit natural light in the manner and intensity best desired?

The cantilevering of the transverse beams for the roof of the Printing Hall is an ingenious piece of design. At first sight it would have been easier to span the beams over two outer rows of columns. Apart from the question of expansion joints is it that

the depth was necessary to accommodate the shells and therefore it was thought best to take advantage of it by cantilevering the beams? Deep beams are terribly 'oppressive' and in the Printing Hall their depth is about half the clearance above the floor. Did the designers take advantage of the continuity of the shells at the beams?"

Arup defended the criticisms with the following arguments:

Whether a symmetrical roof structure is necessarily more beautiful than an asymmetrical one is open to doubt, but I think it would be cheaper. Unfortunately I do not know of any type of glazing which will turn a South light into a North light, but it would certainly be interesting to have more information on this subject.

It is quite right that the cantilever construction in the General Printing Hall was chosen to take advantage of the fact that the depth was necessary to accommodate the shells and to obtain thereby a simplification of expansion joints. The 'oppressiveness' of the beams is diminished by the fact that their bottom flange is practically level with the lower edge of the shells.

The designers did take advantage of the continuity of the shells at the beams by longitudinal pre-stressing.

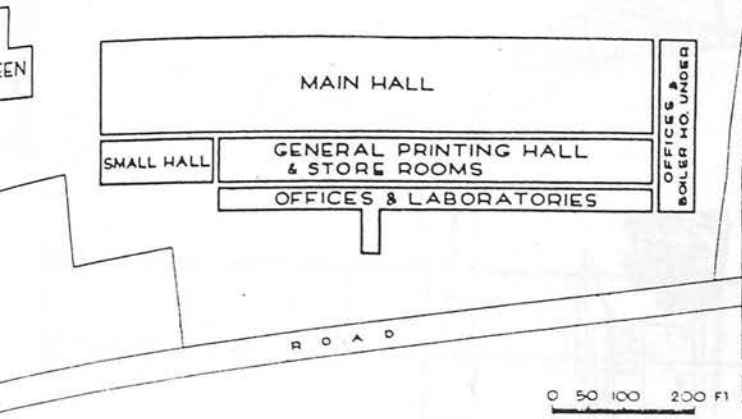


Fig. 2.—Site Layout (diagrammatic)

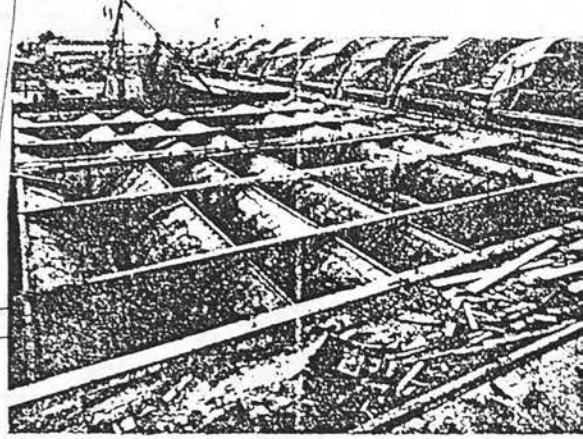


Fig. 3

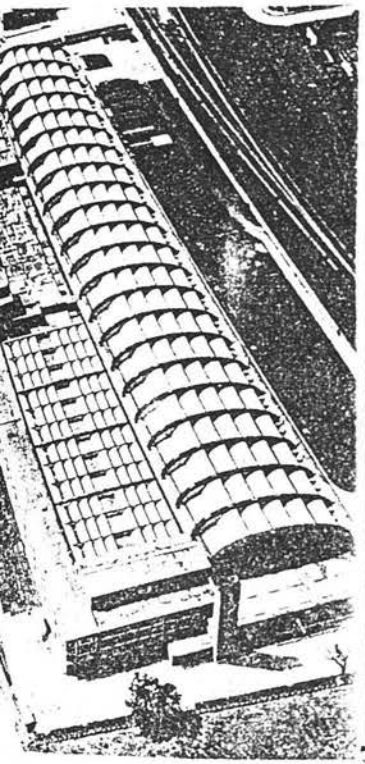


Fig 1

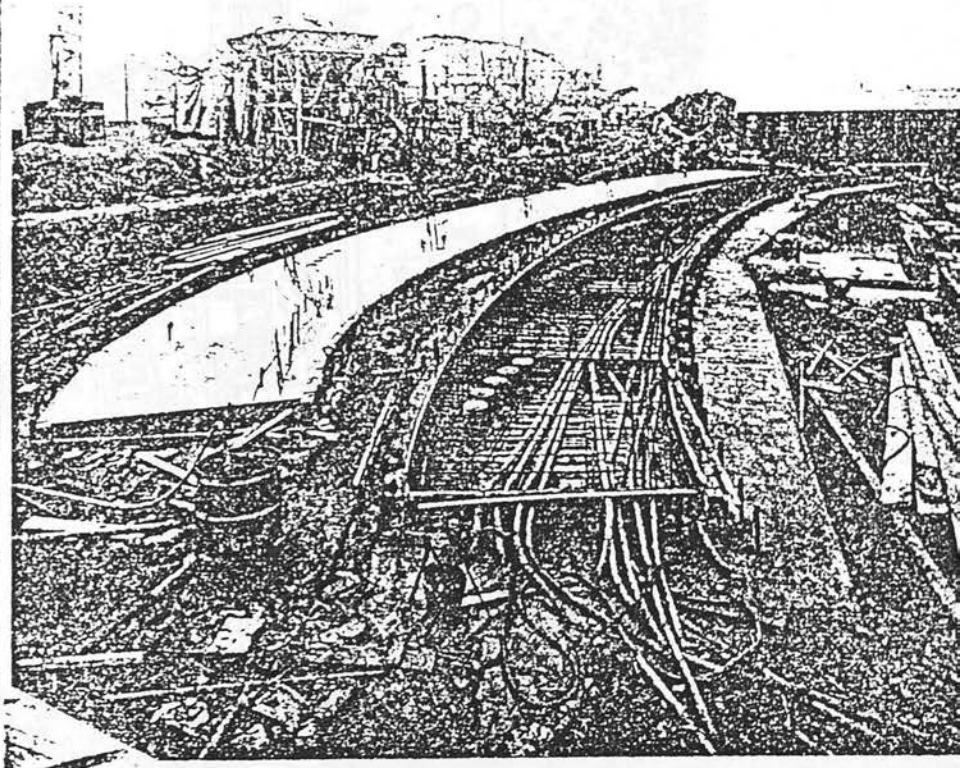


Fig. 4

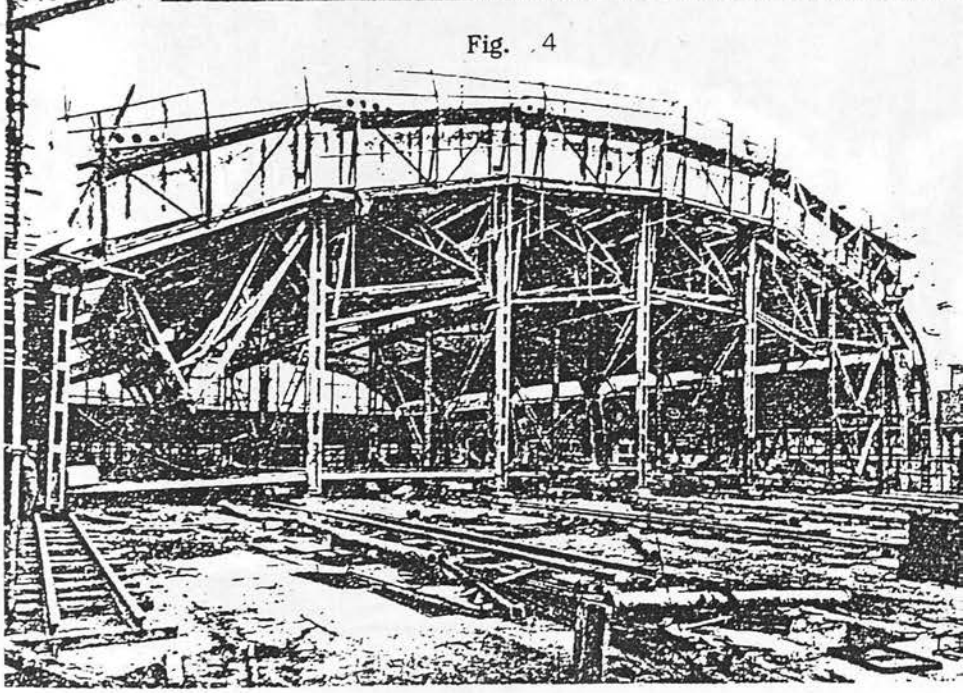


Fig. 5

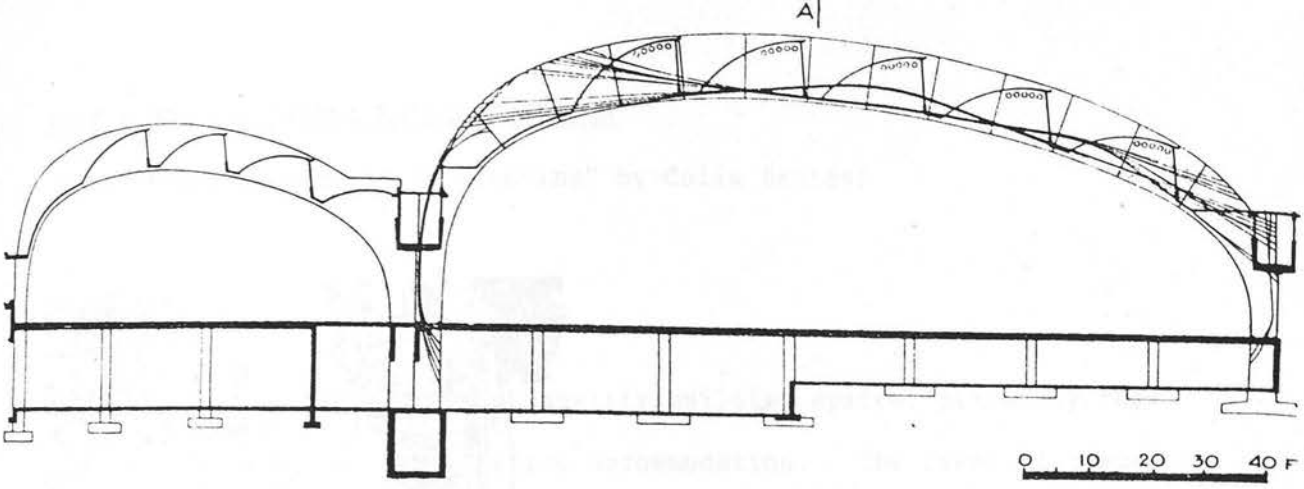
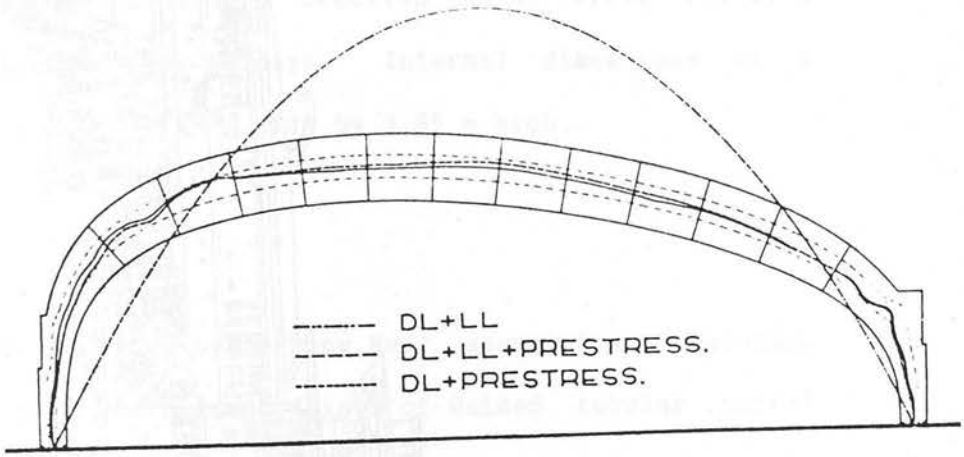


Fig. 6.—Cross section through large and small halls of main production area



SECTION THRO' MAIN PRODUCTION HALL
THRUST LINES OF ARCH SHOWING EFFECT OF PRESTRESSING

Fig. 7

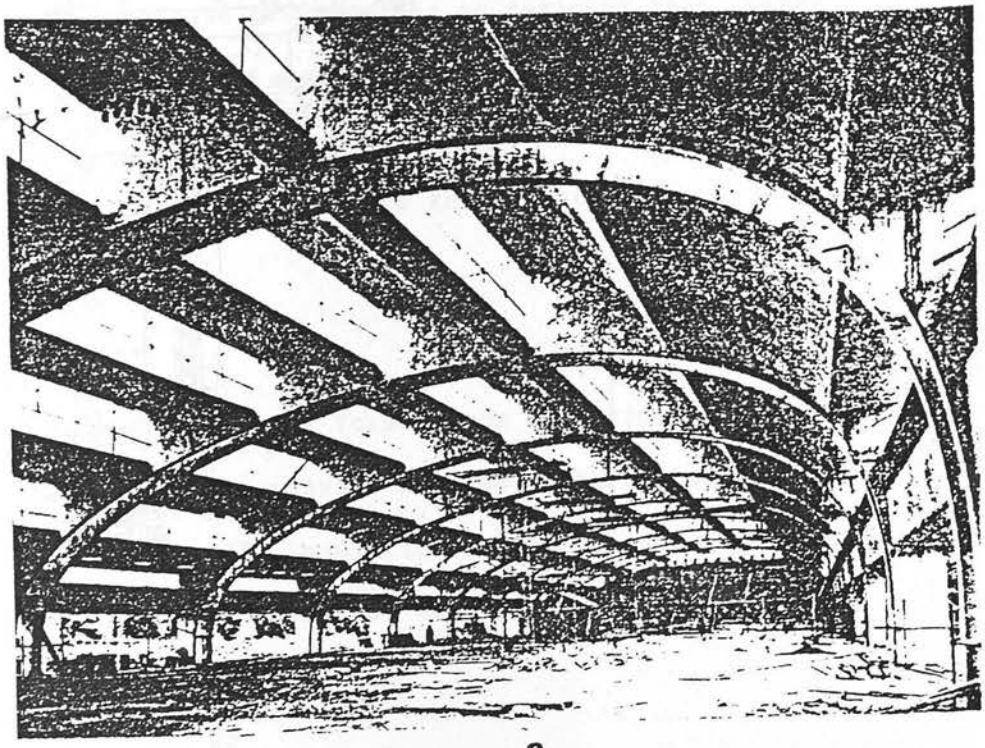


Fig. 8

CASE STUDY 11: PATERA BUILDING SYSTEM

(Extract from "Concepts In Cladding" by Colin Davies)

General

The brief was to develop a high-quality building system, primarily for use as industrial units and office accommodation. The first of these nursery industrial units was erected at Stoke-on-Trent. The building is supplied as a complete package and, apart from the ground floor slab, no wet trades are involved. Erection can be carried out by a small team in approximately ten days. Internal dimensions of a typical unit are 18 m long by 12 m wide by 3.85 m high.

Structure

The unique structure designed by Anthony Hunt Associates, engineers (Project Engineer Mark Whitby), consists of welded tubular portal frame trusses linked by stainless-steel connectors to rectangular hollow-section purlins and angle cross rails, suspended below the lower boom of the trusses. The externally-expressed frame, consisting of a series of centrally-hinged trusses, is so designed that under wind loading comprression is developed in the outer boom but only at the knee and in the legs, which are restrained by a single horizontal tubular tie linking the outside of the truss knee joints.

Cladding

The 3.6 m by 1.2 m cladding panels are formed from two skins of 0.8 mm hot-dip galvanised sheet steel, to provide a boxed construction 150 mm thick filled with mineral-wool insulation during manufacture, constructed to prevent cold bridging and to create an impervious

inner lining. Finish to panels allows for chemical wash to degrease and decontaminate hot-dip galvanised substrate, followed by stoved PVf2 paint system (finish silver grey).

The panels are supported by the 100 mm by 50 mm by 3.2 mm thick rectangular hollow-section purlins with 40 mm by 40 mm by 5 mm thick mild steel angles attached on two sides suspended below and at right angles to the portal frames on 32 mm diameter stainless steel connectors. The panels thus span across their shorter dimension and are formed with regular troughed indentations to provide rigidity. Structural calculations for the external panels were based on the diaphragm action of the outer skin only, which was Z5 deep drawing steel and profiled to a greater depth than the inner skin, which was Z2 galvanised steel strip.

These indentations are stamped into the flat 0.8 mm sheet by three operations on a deep drawing press. Early prototypes suffered from oil canning after pressing and finishing. However, a small additional indentation at the face of each pressing solved this particular problem.

The four edges of the panels are profiled to interlock with the purlins and the two 40 mm by 40 mm by 5 mm angle cross rails running under the trusses.

End walls were constructed using Forster rolled and welded steel frames (section 0153S and 0253S) welded together as ladders (horizontal sections 60 mm, vertical doubled sections 80 mm).

Jointing

The 'tartan' grid of panels fitting into the purlins and cross rails is sealed using ethylene propylene gaskets, by Climax building Gaskets, which fit as a continuous frame, with moulded corners, around the edge of each panel. The EDPM jointing strip was asymmetrical in the final design, to allow for greater cover on the panel side than the capping strip side and to allow a distinct silver line to be visible between gaskets. Purlin covers are used to enclose the purlins from the inside. These are filled with 'Alphire' compressed mineral-fibre rigid board, to maintain thermal insulation and fire protection.

Construction

In order for this form of press-in joint gasket system to work, the reader will appreciate that a very high degree of accuracy is required from the assembly. Erection tolerances are quoted as +/- 1 mm for the overall dimensions of the building; structural fabrication tolerances as +/- 0.5 mm, component tolerances as +/- 0.25 mm, and gaskets are manufactured to accommodate tolerances/movement in a range of + 1 mm/- 3 mm.

In order to obtain these high standards of accuracy, it was necessary to pre-jig the frame and panels in the factory. Trusses were set out in welded jigs and the positions of the stainless steel connectors accurately located. Purlins were also fabricated to exact lengths and pre-drilled using jigs. These act as a template to co-ordinate the structure and the panels on site and enable fine tolerances to be achieved. Holes for fixing bolts to the anchor plates are drilled with marked positions on site. Pre-positioned anchor bolts are not

required.

11

CASE STUDY: PATERA BUILDING SYSTEM

(Extract from "Concepts In Cladding" by Colin Davies)

General

The brief was to develop a high-quality building system, primarily for use as industrial units and office accommodation. The first of these nursery industrial units was erected at Stoke-on-Trent. The building is supplied as a complete package and, apart from the ground floor slab, no wet trades are involved. Erection can be carried out by a small team in approximately ten days. Internal dimensions of a typical unit are 18 m long by 12 m wide by 3.85 m high.

Structure

The unique structure designed by Anthony Hunt Associates, engineers (Project Engineer Mark Whitby), consists of welded tubular portal frame trusses linked by stainless-steel connectors to rectangular hollow-section purlins and angle cross rails, suspended below the lower boom of the trusses. The externally-expressed frame, consisting of a series of centrally-hinged trusses, is so designed that under wind loading comprression is developed in the outer boom but only at the knee and in the legs, which are restrained by a single horizontal tubular tie linking the outside of the truss knee joints.

Cladding

The 3.6 m by 1.2 m cladding panels are formed from two skins of 0.8 mm hot-dip galvanised sheet steel, to provide a boxed construction 150 mm thick filled with mineral-wool insulation during manufacture, constructed to prevent cold bridging and to create an impervious

inner lining. Finish to panels allows for chemical wash to degrease and decontaminate hot-dip galvanised substrate, followed by stoved PVf2 paint system (finish silver grey).

The panels are supported by the 100 mm by 50 mm by 3.2 mm thick rectangular hollow-section purlins with 40 mm by 40 mm by 5 mm thick mild steel angles attached on two sides suspended below and at right angles to the portal frames on 32 mm diameter stainless steel connectors. The panels thus span across their shorter dimension and are formed with regular troughed indentations to provide rigidity. Structural calculations for the external panels were based on the diaphragm action of the outer skin only, which was Z5 deep drawing steel and profiled to a greater depth than the inner skin, which was Z2 galvanised steel strip.

These indentations are stamped into the flat 0.8 mm sheet by three operations on a deep drawing press. Early prototypes suffered from oil canning after pressing and finishing. However, a small additional indentation at the face of each pressing solved this particular problem.

The four edges of the panels are profiled to interlock with the purlins and the two 40 mm by 40 mm by 5 mm angle cross rails running under the trusses.

End walls were constructed using Forster rolled and welded steel frames (section 0153S and 0253S) welded together as ladders (horizontal sections 60 mm, vertical doubled sections 80 mm).

Jointing

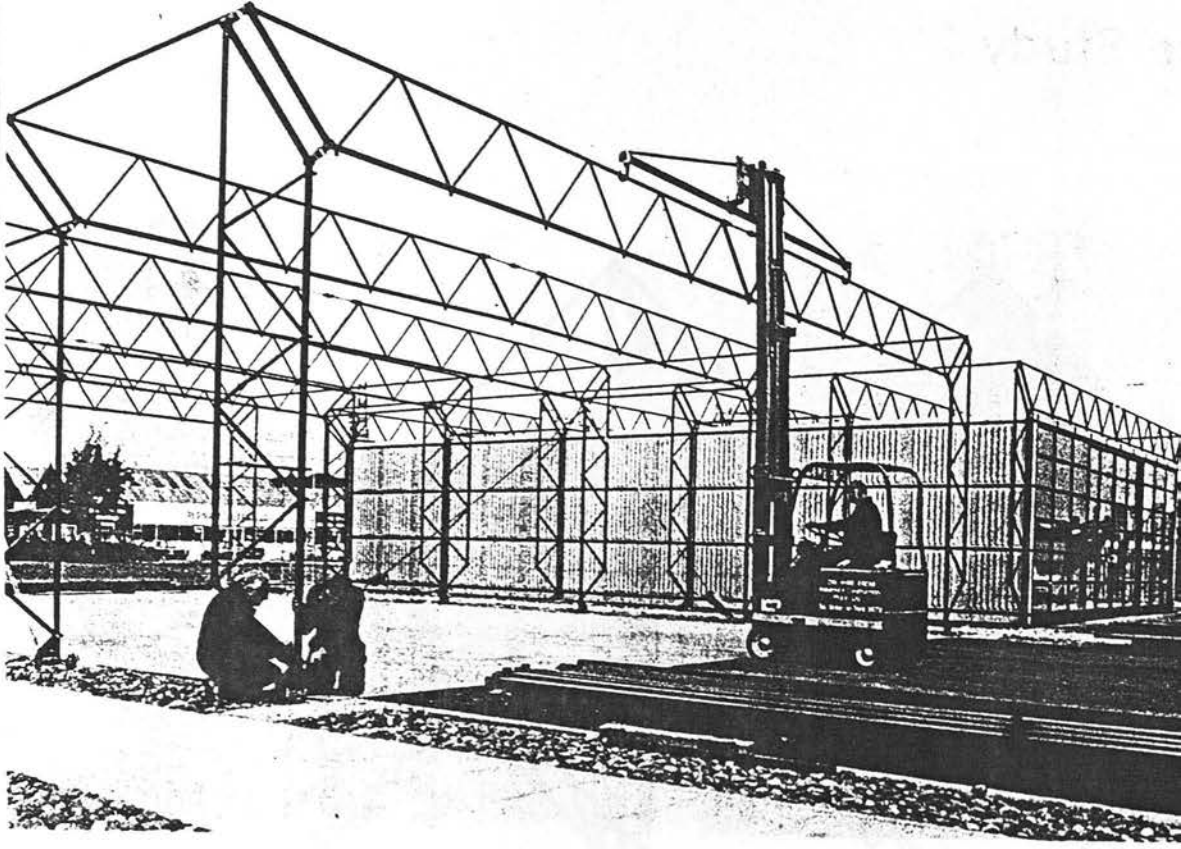
The 'tartan' grid of panels fitting into the purlins and cross rails is sealed using ethylene propylene gaskets, by Climax building Gaskets, which fit as a continuous frame, with moulded corners, around the edge of each panel. The EDPM jointing strip was asymmetrical in the final design, to allow for greater cover on the panel side than the capping strip side and to allow a distinct silver line to be visible between gaskets. Purlin covers are used to enclose the purlins from the inside. These are filled with 'Alphire' compressed mineral-fibre rigid board, to maintain thermal insulation and fire protection.

Construction

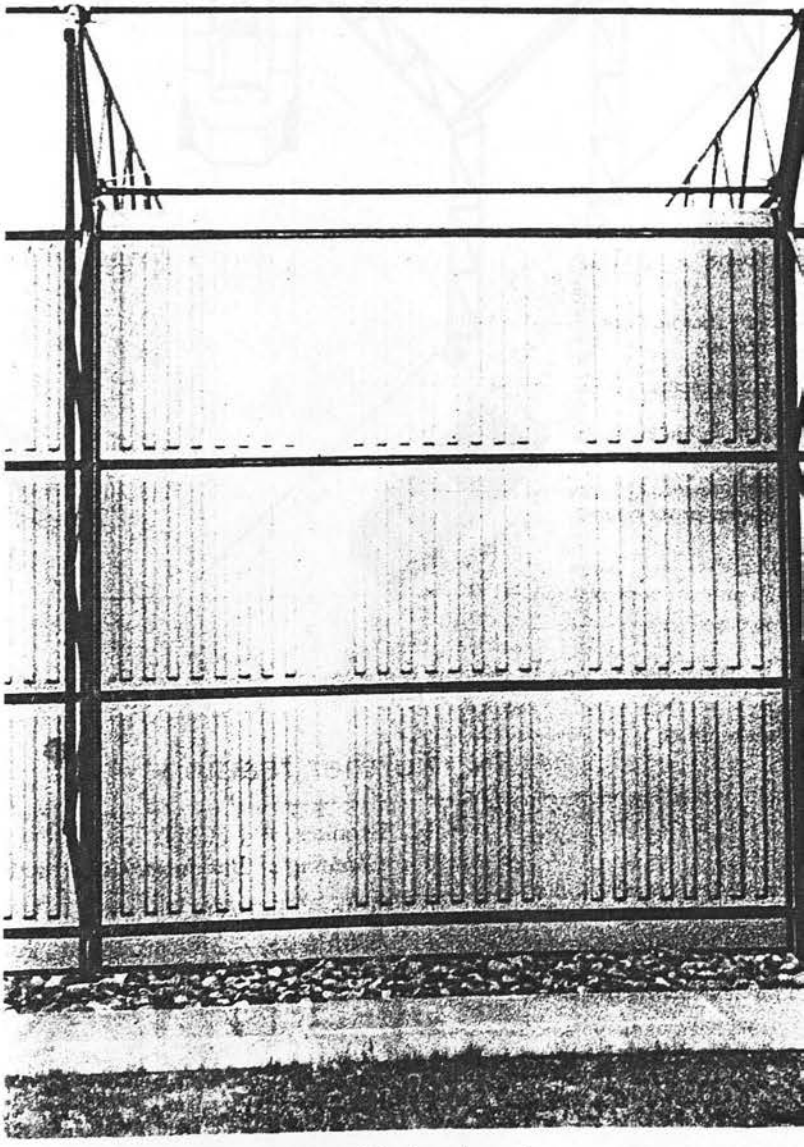
In order for this form of press-in joint gasket system to work, the reader will appreciate that a very high degree of accuracy is required from the assembly. Erection tolerances are quoted as +/- 1 mm for the overall dimensions of the building; structural fabrication tolerances as +/- 0.5 mm, component tolerances as +/- 0.25 mm, and gaskets are manufactured to accommodate tolerances/movement in a range of + 1 mm/- 3 mm.

In order to obtain these high standards of accuracy, it was necessary to pre-jig the frame and panels in the factory. Trusses were set out in welded jigs and the positions of the stainless steel connectors accurately located. Purlins were also fabricated to exact lengths and pre-drilled using jigs. These act as a template to co-ordinate the structure and the panels on site and enable fine tolerances to be achieved. Holes for fixing bolts to the anchor plates are drilled with marked positions on site. Pre-positioned anchor bolts are not

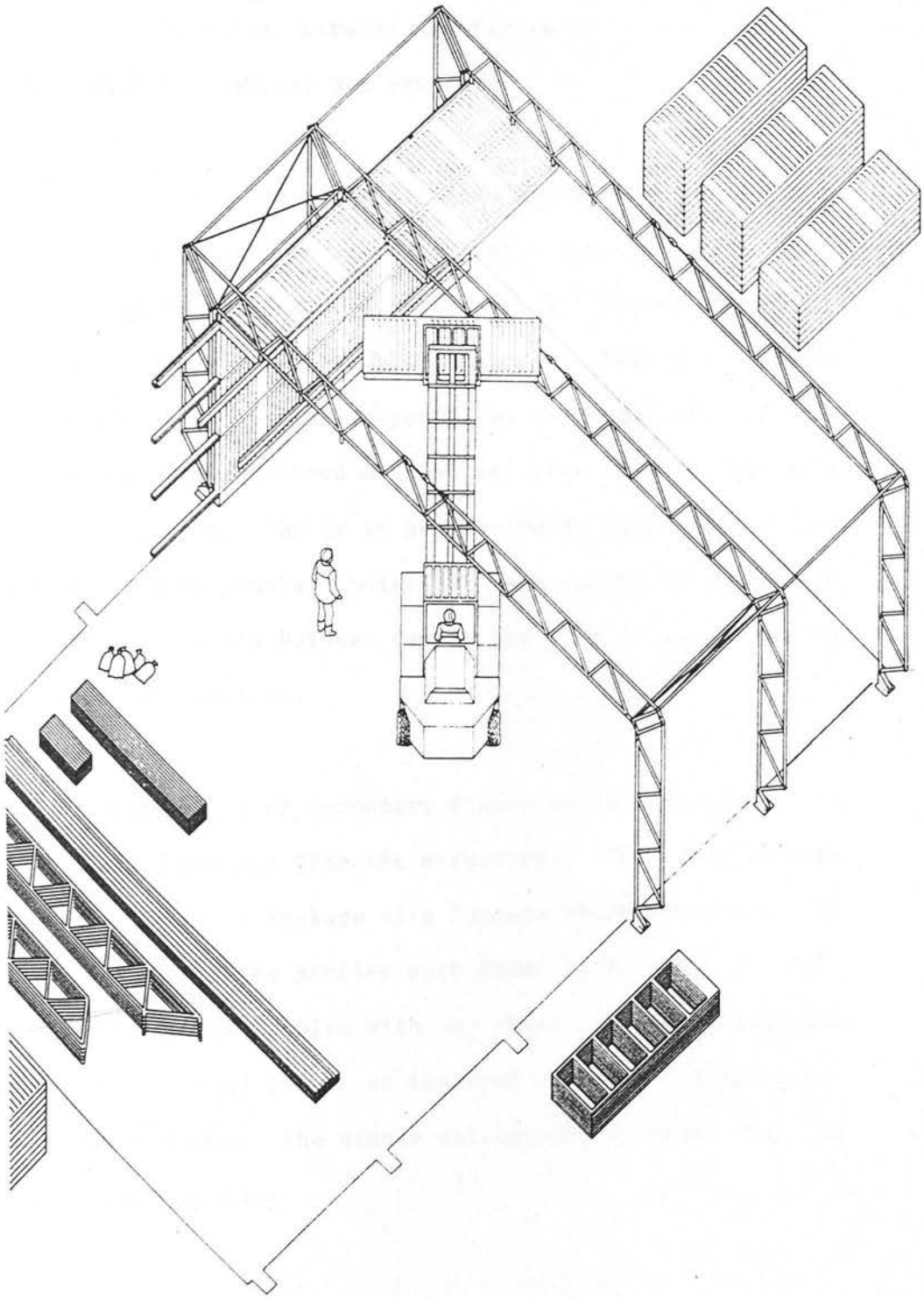
required.



Structure under assembly (Courtesy of *Architects' Journal*; photo by Dave Bower)



General view showing ribbed steel panels



Patera system — assembly of panels (Courtesy of Michael Hopkins (Architects))

CASE STUDY 12: IBM SPORTS HALL

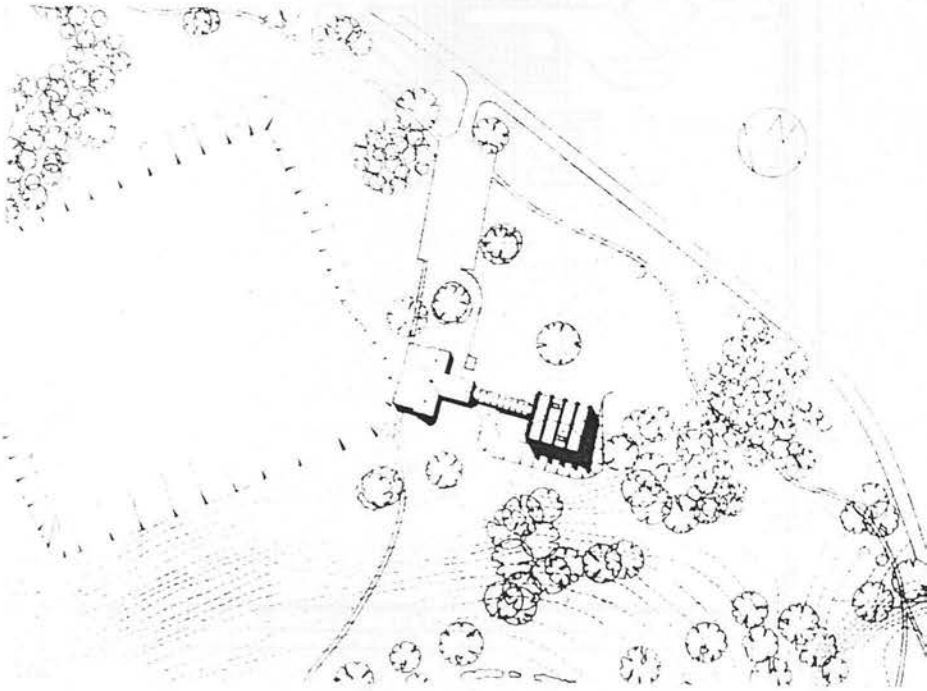
(Extract from a Criticism by John Winter, AR, March 1982)

Architect : Nicholas Grimshaw and Partners

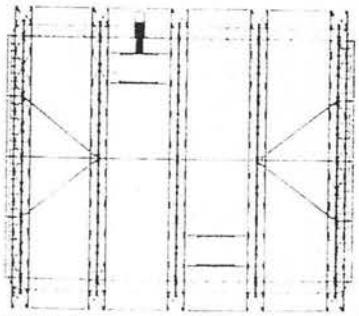
Structural Engineers: F J Samuely and Partners

The roof structure spans 18 m and it essentially comprises five trussed-portals (triangulated in section) braced longitudinally with diagonal members (88 mm x 5 m x 10.3 kg/m circular hollow section). The portals are two-pinned on steel base brackets, forked to accept the trussed portals. the portals support the roof and walls of the Sports Hall, and they are conceived as five separate planes. The wall panels 5250 mm in length, 500 mm in height and 42 mm in depth and comprise industrial door panels, bolted at each corner to cleats on the frame. Vertical joints between panels are a 15 mm gap inserted with a neoprene gasket, caulked.

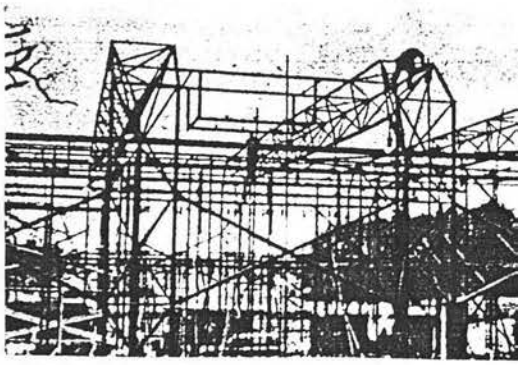
There are no cladding rails or secondary frames in this system. The roof is a flat plane suspended from the structure. The disadvantage lies in the possibility of leakage with hangers which penetrate the roof membrane. This is the problem with Mies' Crown Hall at IIT. Grimshaw has resolved this problem with pvc sheet roof covering and pre-formed upstands of rigid pvc. As the roof membrane and dressings are of the same ~~membrane~~ material, the single waterproof covering may be tailored to the required shape.

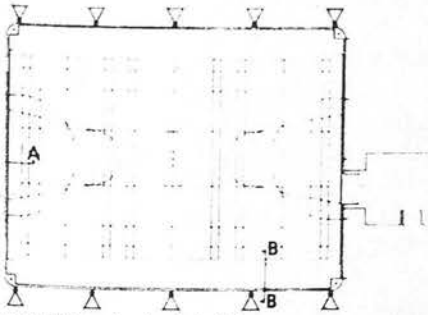


site plan

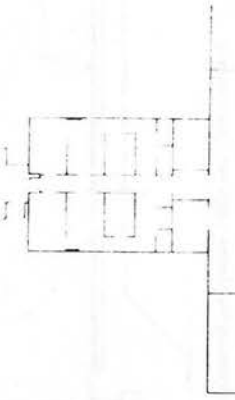


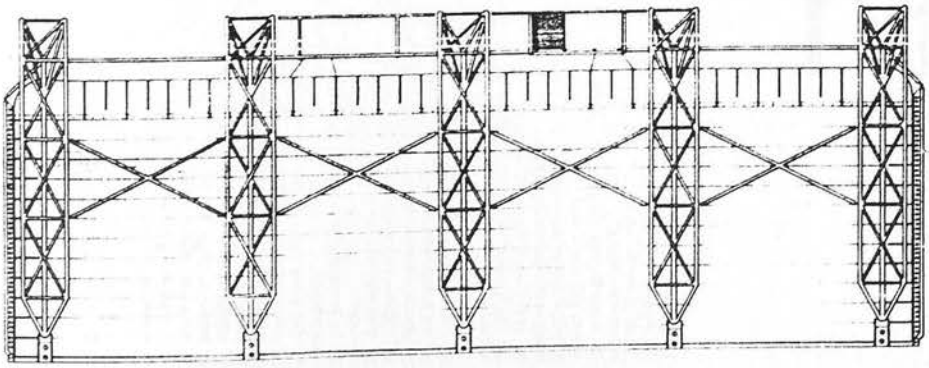
roof plan





ground floor plan (scale 1:500)

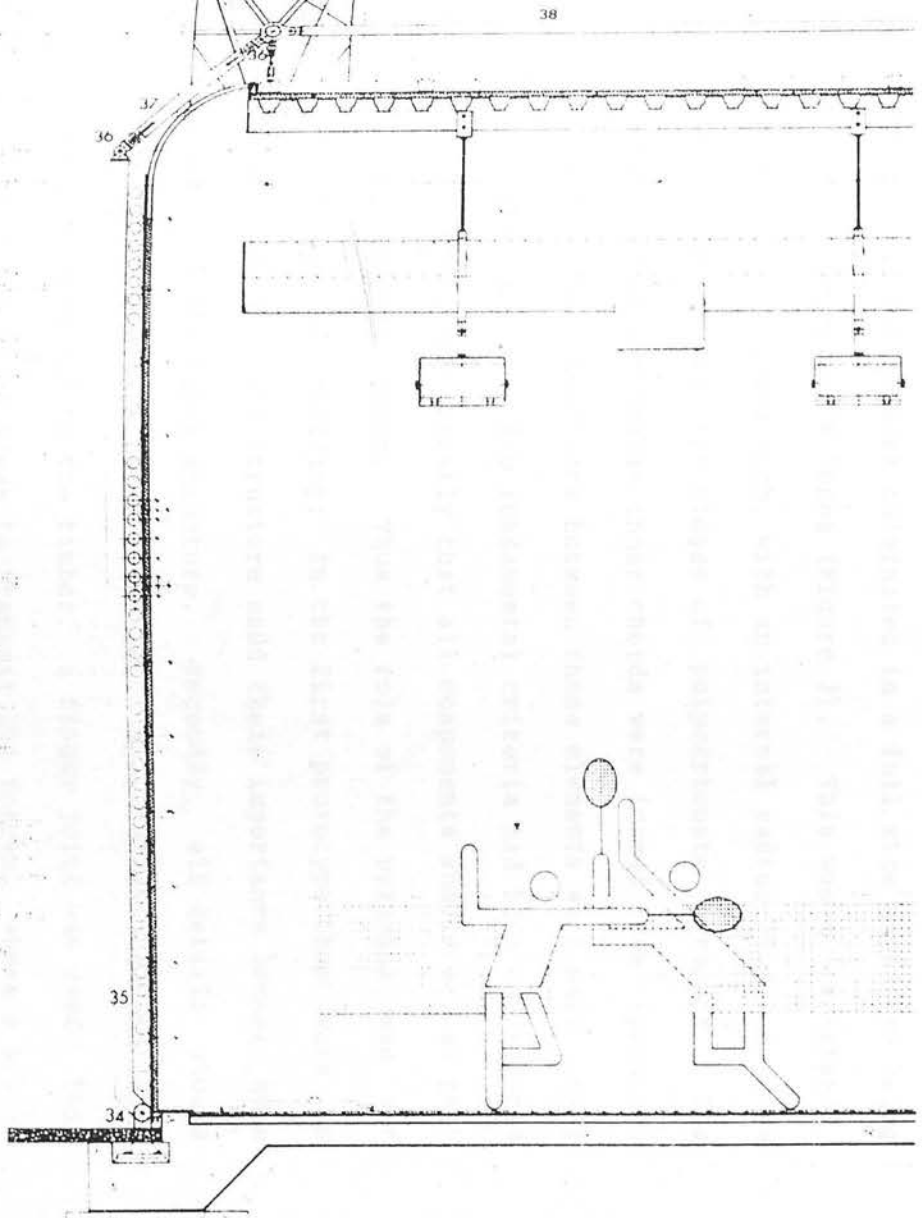




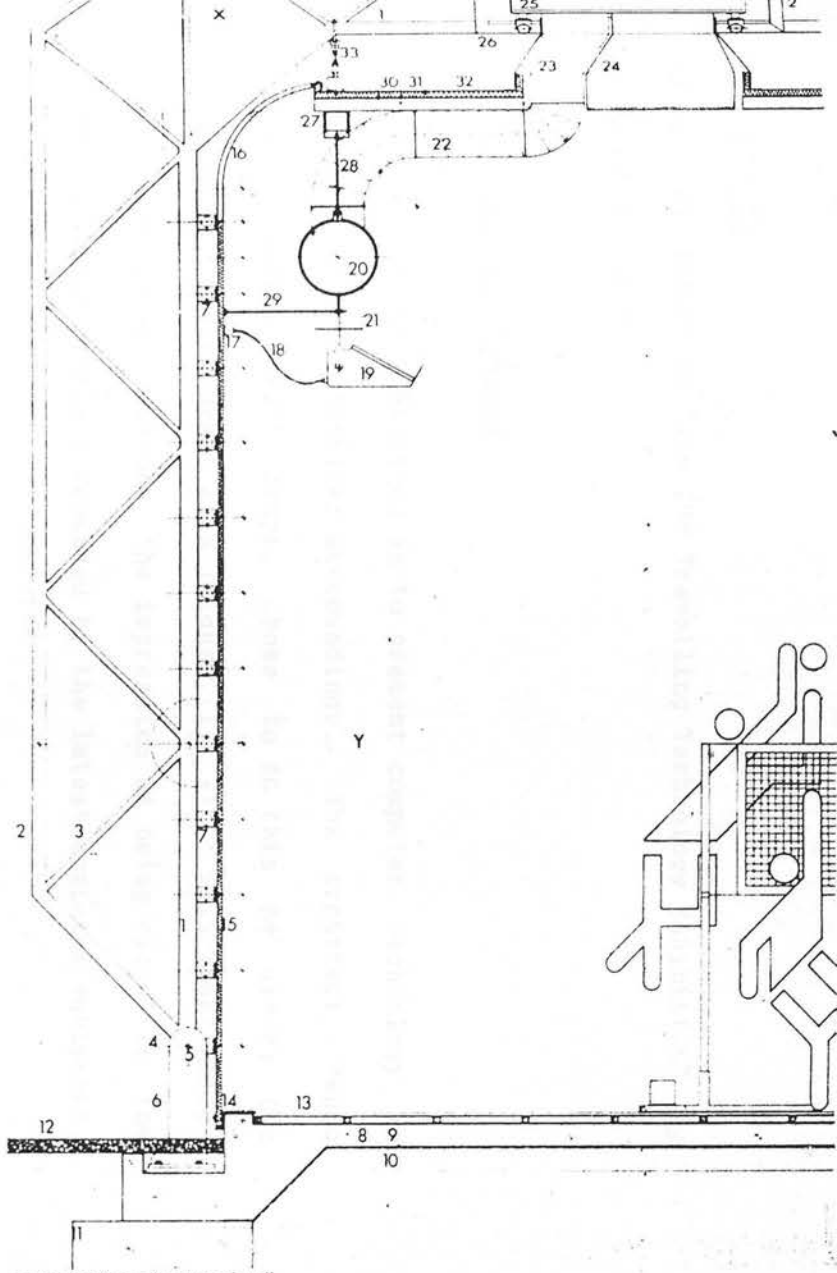
elevation

section
3. 48·3 mm dia × 4 [illegible]
section
4. 244·5 mm [illegible]
76 mm × 22 × [illegible]
[illegible]
5. [illegible]
[illegible]
6. steel base [illegible]
pin joint
7. cladding cleats
146 mm × 127 [illegible]
100 mm long with [illegible]
8. reinforced 150 mm [illegible]
9. polythene dpm
10. 150 mm selected hardcore
11. mass concrete foundation
12. gravel surround
13. 22 mm thick [illegible]
flooring laid on [illegible]
battens on [illegible]
14. beech edge [illegible]
15. 42 mm thick [illegible]
cladding [illegible]
16. prefabricated [illegible]
grip curved [illegible]
42 mm thick by 1000 mm [illegible]
17. 50 mm × [illegible]
18. red curly flex [illegible]
19. indirect flood lights
20. 500 dia spiral wound [illegible]
21. drum type [illegible]
22. supply air ducts
23. supply air
24. return air
25. electric heat pump (heating [illegible]
26. heat pump cradle support
27. 150 mm × 150 mm [illegible] rectangular
section purlin
28. air handling duct bracket and hanger
assembly
29. duct restraining [illegible]
30. 100 mm profiled steel deck [illegible]
31. 50 mm rigid insulation board
32. pvc sheet weatherproofing [illegible]
33. roof hanger assembly

end mullions
34. 139·7 mm dia × [illegible]
35. 114·3 mm dia × 5·0 [illegible]
continuous welded to a 146 [illegible]
22 kg/m structural tee with [illegible]
web at 125 mm centres
36. nodus pin joint connection [illegible]
centre tube (type 4)
37. 88·9 mm split [illegible]
38. wind bracing: 88·0 [illegible]
10·3 kg/m circular hollow [illegible]



section A-A through external wall (scale 1:50); see plan overleaf



section B-B through external wall

CASE STUDY 13

(Extract of Report on "The IBM Travelling Technology Exhibition" by Robert Kinch (OAP))

1.0 Architectural Concept

The aim of the exhibition is to present computer technology to young people in familiar surroundings. The architect, Renzo Piano, working with Arups, chose to do this by giving the building a transparent skin (Figure 1, 2). This see-through building gives visitors the impression of being close to the natural world, while surrounded by the latest computer equipment.

The initial design work culminated in a full size prototype being built on the beach in Genoa (Figure 3). This was a two-pinned, triangular, trussed arch, with an internal radius of 4.9 m. The web members were the ridges of polycarbonate pyramids. The single outer and double inner chords were formed from laminated timber. The connections between these elements were made using aluminium nodes. Two fundamental criteria had been established in this prototype: firstly that all components should act as part of the support system. Thus the role of the pyramids was not simply one of cladding; in the first prototype they were an integral part of the structure andd their importance became even greater in the final structure. Secondly, all details should clearly demonstrate their function. Where aluminium connecting pieces were bonded to the timber, a finger joint was used. This allowed large glued areas to transmit the forces, where a bolted detail would have resulted in much larger timber sections.

2.0 Ventilation

The exhibition comes complete with its own air conditioning system. There are six free-standing air-handling units mounted along the centre line of the exhibition, with two further units positioned above the doors in the double-skinned walls at each end of the building. The floor mounted units supply air to the floor ducts which discharge via perimeter grilles and circular twist grilles mounted in the floor. The units in the end walls supply fresh air to the exhibition and to a 'spine' duct which runs the length of the building at high level, supported by the crown of each arch. From this, vein ducts extend at each arch to arch junction. These follow the curve of the arch and vent directly onto the internal surfaces of the top four pyramids on each side of the crown, thus inhibiting the formation of condensation.

3.0 The Influence of Polycarbonate

The most important factors affecting the design decisions for both services and structure had their roots in the physical and mechanical properties of polycarbonate. It has a very high coefficient of linear expansion, a low stiffness, which decreases rapidly with increasing temperature, it is extremely transparent and has a low thermal insulation value.

In the original prototype, the pyramids were fixed rigidly to the timber members. In hot conditions, when the surface temperature can reach 85 ^{o}C, the compressive forces in the pyramids resulting from restrained expansion would cause buckling of the polycarbonate. Early attempts to overcome this involved

installing a spring between the pyramids and the inner timber chords. This was to allow the pyramids to move radially but not cirrcumferentially, taking up dimensional variations by changing the radius of the pyramids. It had been decided to separate the timber inner chords and the polycarbonate by 2000 mm. Using a shear-stiff spring to achieve this articulation led to relatively high bending movements where it was fixed to the pyramids and timber. To increase the strength of the components to overcome this would have complicated an already involved manufacturing process and so the spring was abandoned.

The next solution studied, and the one finally adopted, was to use an axially stiff connecting rod pinned at each end (Figure 6). This allowed the pyramids to expand circumferentially relative to the internal timber chords. Variations in the dimensions of the polycarbonate were absorbed by changes in the overall arch radius. The fixing to the pyramids allowed rotation in the direction of the building's long axis. This allowed shear forces from longitudinal wind loads to mobilise the vierendeel action of the inner timber frame.

A consequence of the pin-ended rod was that under asymmetric loads, the inner timber members received radial loads only. Their stiffness acting as an arch was less than the truss formed by the outer timber chord and the pyramids. The inner edges of the pyramids thus became the inner chords of the truss. With the pyramids now forming both web and inner chord members, the low stiffness of the polycarbonate led to the introduction of arch to arch connections and stiffening of the chord elements of the pyramids.

The forces acting on the inner edges of the pyramids were sufficient to cause them to buckle. It was originally proposed to stiffen the edges by gluing a channel-shaped polycarbonate extrusion to them. In practice this proved very difficult. It was not possible to achieve true mating surfaces and the only transparent glue available caused extensive stress cracking of the polycarbonate. The extrusions were replaced by the aluminium channels, one outside and the other inside the pyramid edges. These were bolted together through the polycarbonate, slotted holes in the aluminium allowing differential thermal movements. The channels serve to prevent buckling only and carry no axial loads (Figure 11).

Analysis of the structure showed that deflections in both radial and longitudinal directions, under design loads, would be large. The longitudinal deflections of a single arch under wind load were too great to be acommodated by the water-proofing details. These deflections were accompanied by racking of the arch. By connecting the arches together at the crown and the mid-point on each side, the racking was minimised and the deflection reduced to an acceptable level. The connections were made between the aluminium nodes in the inner timber members. They serve tto transmit shear between arches and also form supports for the vein ducts (Figure 12).

Radial deflections, although large, were not detrimental to individual arches. They did, however, require careful detailing of the joint between the end walls and the arches enclosing them. If these arches deflected to the point at which they received support from the walls, large shear forces would be generated in

the arch to arch connections. To prevent this occurring, the wall panels and the arches were separated by 150 mm. This grap was then sealed by fixing flat polycarbonate sheets to the arch, allowing them to slide relative to the walls. A flexible gasket between these panels and the pyramids absorbs differential longitudinal movements.

4.0 The Influence of Transporation and Assembly

The need to transport and erect the exhibition was the second most important factor affecting the design. For transport, each arch is separated into four major components (See Figure 16):

i) Two outer chord members.

ii) Four inner chord members.

iii) 14 inner chord cross members.

iv) Four sets of pyramids.

It was only after the components had been decided upon that joints could be designed. Where possible misalignment of members could happen without affecting the ease of assembly, bolted connections were adopted. For those joints where misalignment due to temperature and tolerance effects could make assembly difficult, a more tolerant type of connection was developed. Those were the joints between the timber members and the stainless steel connecting pieces. Here tapered rubber blocks were cast onto the steel; these allowed a push-fit even when perfect match was not achieved. The joints were secured using bolted stainless steel pressure plates after assembly.

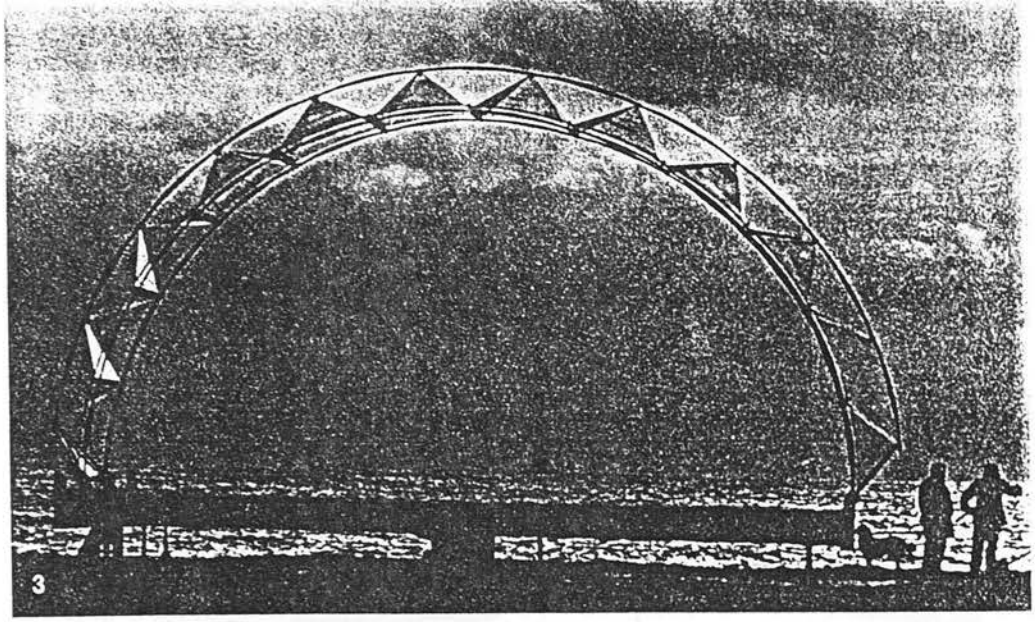
It was decided very early in the design process that no site connections would be made directly to the polycarbonate. This was because it was felt that any structural connection would rely upon shear transfer and require a 'friction grip' type of connection. The level of site supervision needed to achieve this would be unreasonably high. In addition, if this type of connection were made in the factory, periodic checking would be required, as the polycarbonate would creep under sustained pressure. For these reasons, all bolted connections to the pyramid were rejected and glued junctions adopted.

Each of these components can be manhandled, the number of men being determined more by size of component than weight. The way in which each arch is divided up was determined by comparing the number of trucks required with the time needed for assembly. Transportation became the dominant influence in every case except one. Single pyramids could be stacked more economically. However the joint between pyramids was time-consuming and expensive to make and could lead to tolerance misalignment between arches. During manufacturing, many more components were needed to be fixed to the pyramids; the number of specially made connecting pieces used during assembly would have been increased fivefold. Perhaps most importantly of all, the number of joints requiring weather-proofing would also increase fivefold. For these reasons it was decided to form the pyramids in the largest number possible.

This was determined as three by the maximum size of polycarbonate sheet available.

An advantage of this was that the manufacturer of the pyramids was able to use just two moulds; one for the top set of pyramids and one for the bottom.

This ensured that all sets of pyramids were dimensionally compatible (See Figure 17).



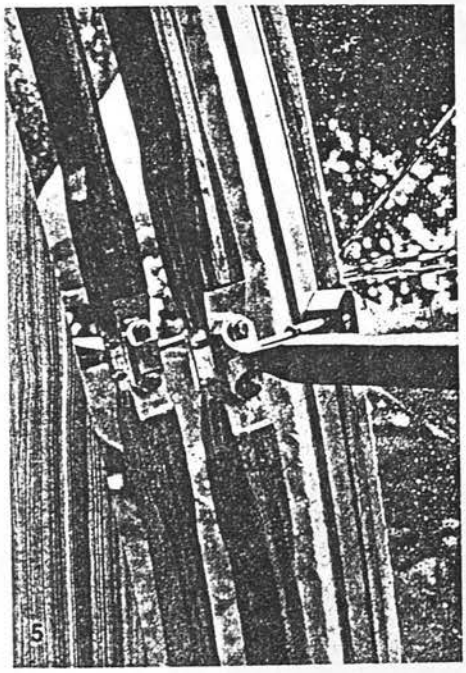


Fig.2
The transparent skin of the building
(Photo: Harry Sowden)

Fig.3
The original prototype
(Photo: Building Workshop)

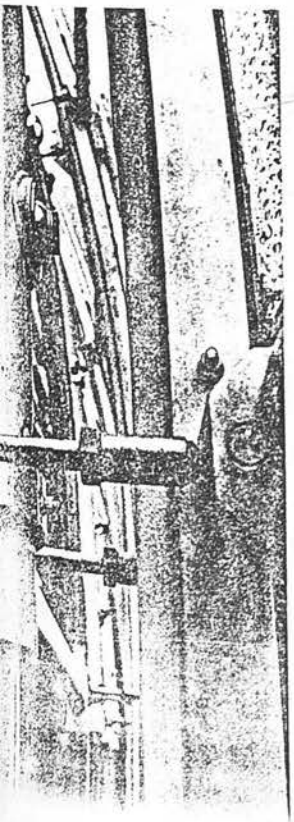
Fig.4
The arch during assembly
(Photo: the architect)

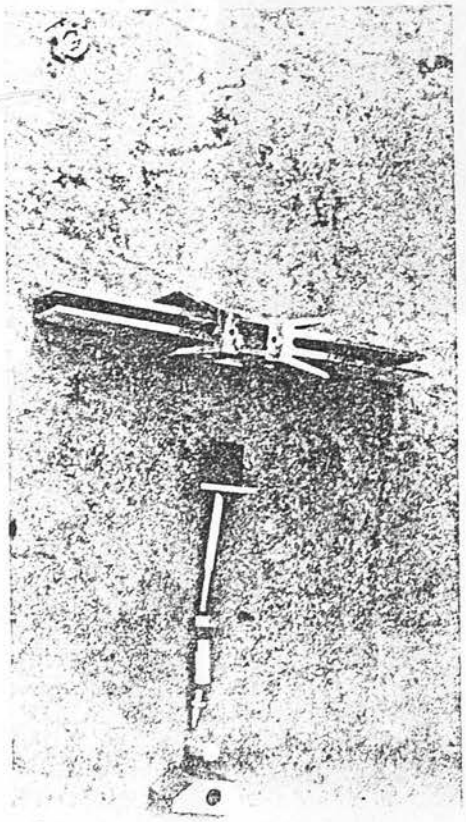
Fig. 5
Typical timber aluminium joint
(Photo: Harry Sowden)

Fig.6
Pyramid-timber connection rod
and anti-condensation air nozzle
(Photo: Harry Sowden)

Fig.7
Connection joint assembly
(Photo: Robert Kinch)

Fig.8
Bottom joint between
external timber and pyramid
(Photo: the architect)





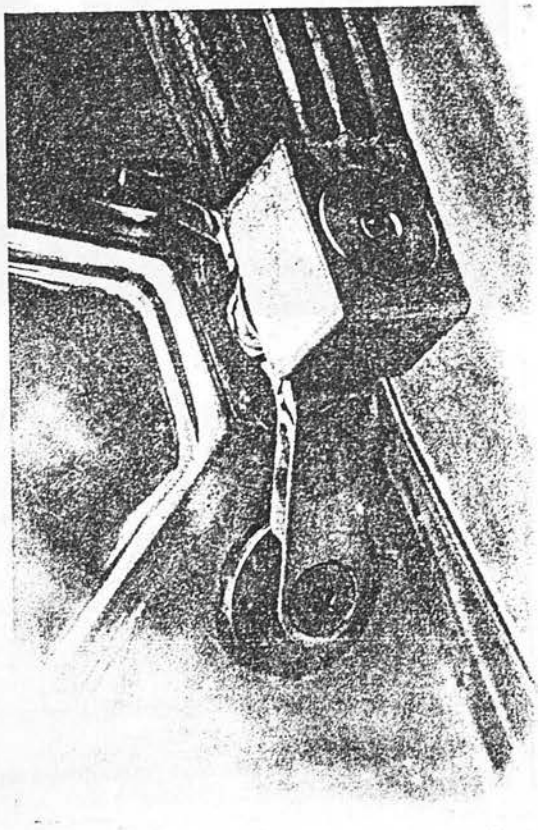


Fig.9
Insulating pyramids
(Photo: Harry Sowden)

Fig.10
Perforated aluminium panels
(Photo: Robert Kinch)

Fig.11
Bottom fixing detail of arch showing internal aluminium stiffening
(Photo: the architect)

Fig.12
Connection between arches
(Photo: Robert Kinch)

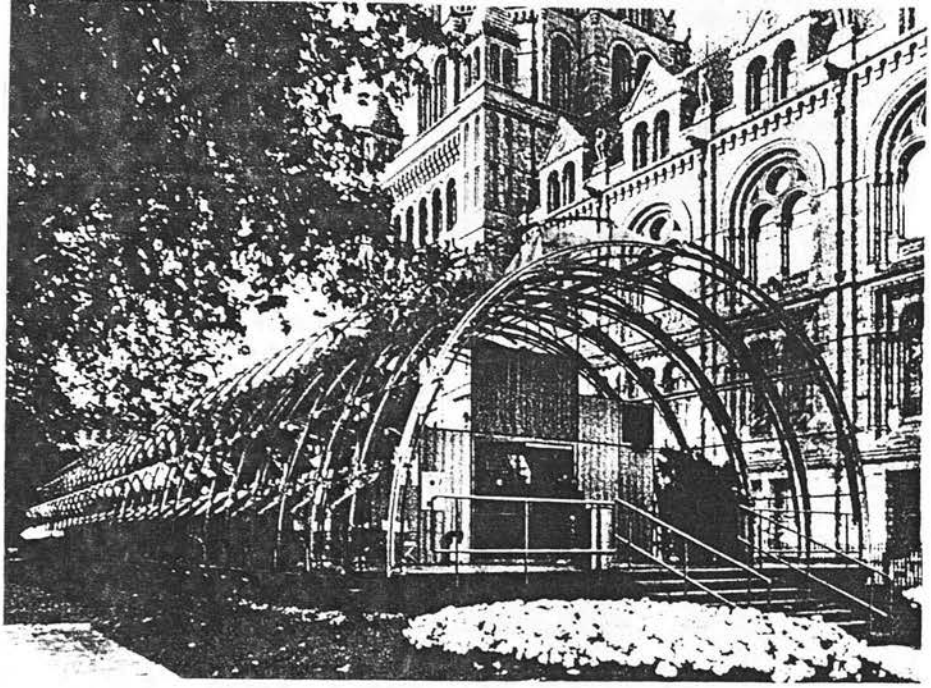


Fig.1

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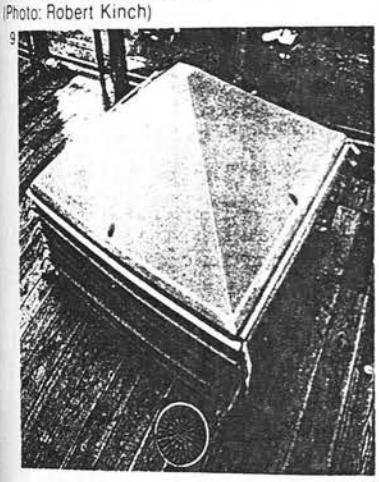

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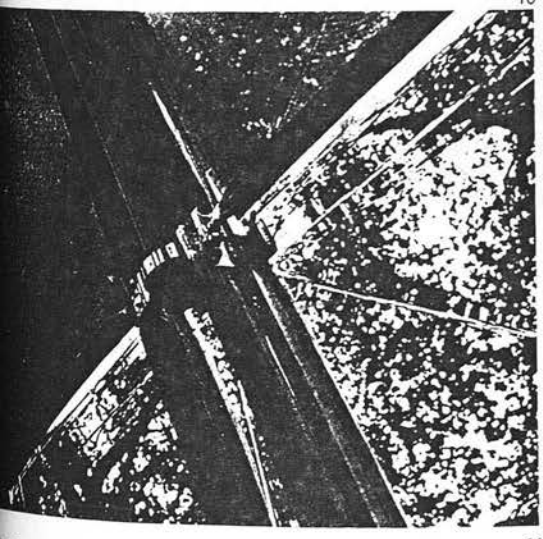

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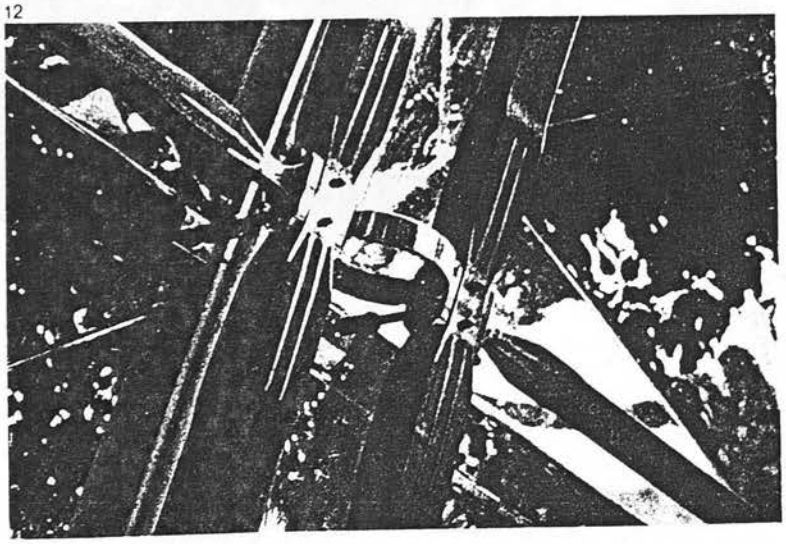

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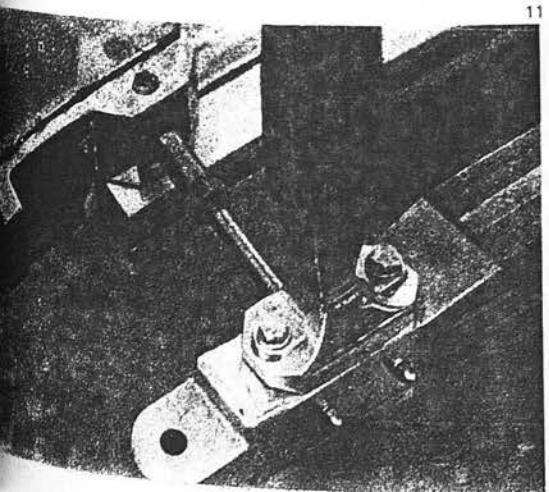


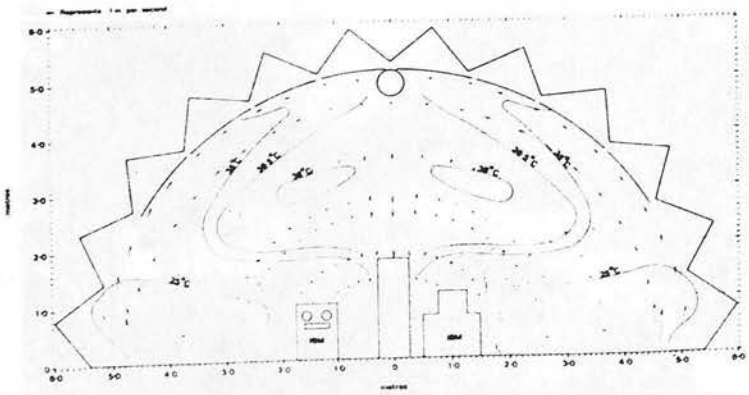
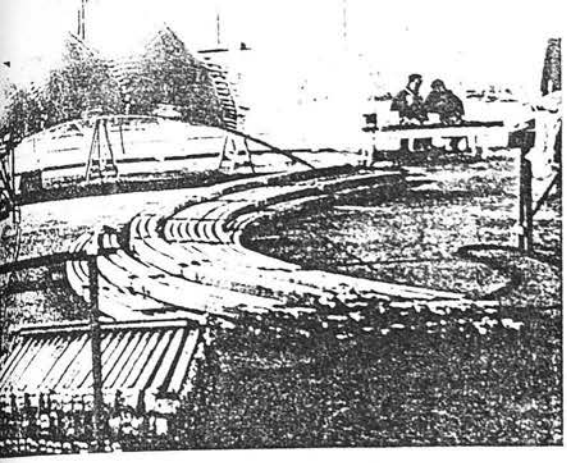


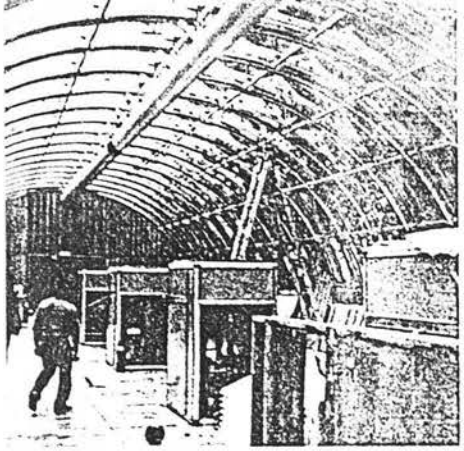
Fig.13
Phoenics simulation of air stratification on a summer day



Arch components in the factory during fabrication (Photo: Robert Kinch)

Fig.18
Air conditioning units during assembly (Photo: Robert Kinch)

Fig.19
The exhibition at night (Photo: Harry Sowden)



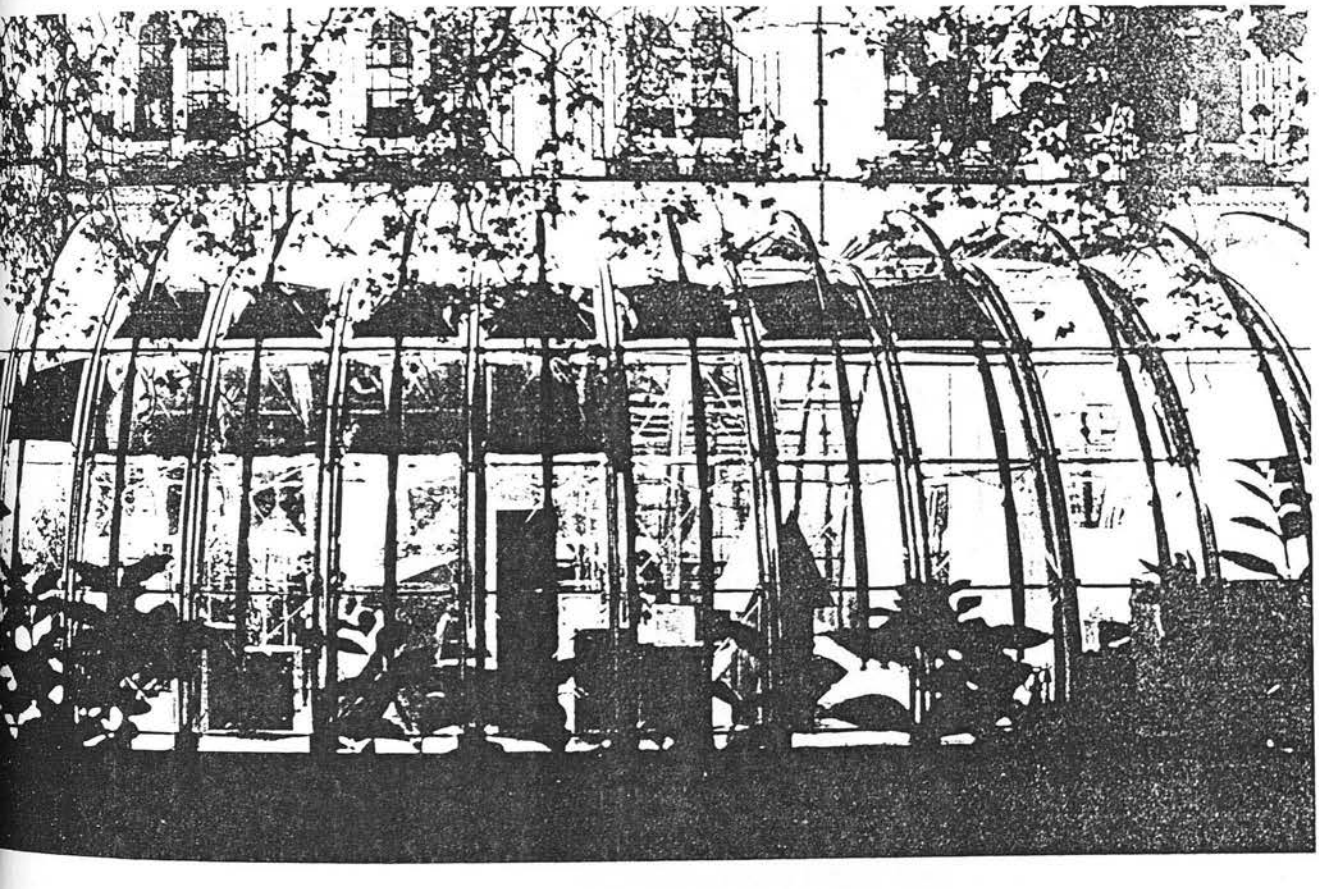


Fig.15
General view of interior showing spine and vein ducts with insulating devices positioned (Photo: Harry Sowden)

Fig.16
Arch components in London: inner and outer timber chords with half arches (Photo: Robert Kinch)

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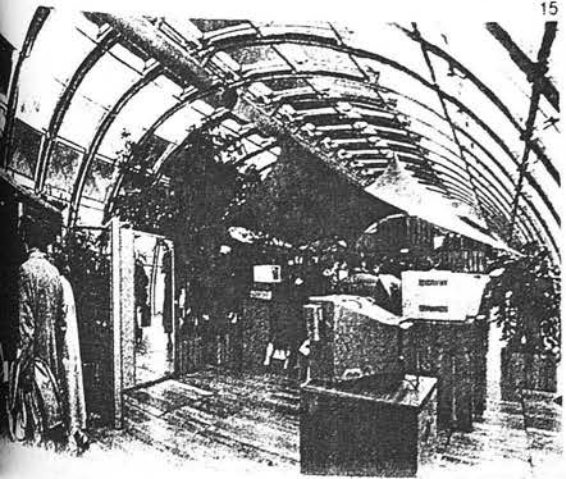

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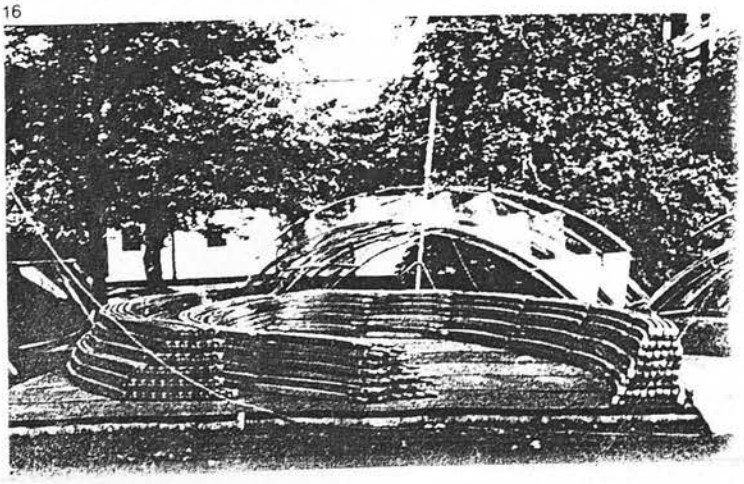

GROUP 2

CASE STUDY No. 14

Extracts from
CUMMINS ENGINE CO. - SHOTTS FACTORY DEVELOPMENT

Extracts from a report by
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1.0 Synopsis

Interest in the architecture and engineering of new industrial buildings continues to grow. Underlying the design of Cummins' latest diesel engine factory was a vigorous search for a natural and clear architectural expression of its many functions. This involved the orchestration of a large multidisciplinary design team. Generous flexibility of layout, rigorous planning for structure/services integration, consistent structural detailing, and concern for progressive collapse, all affected the design, which is described here from the structural engineer's point of view.

2.0 The Project

The Client

Cummins Engine Co. Ltd is part of an American-based multinational corporation, manufacturing diesel engines. They have three main plants in the UK - at Darlington, Daventry, and Shotts in Lanarkshire. This project concerns the expansion of the Shotts plant to double its production capacity.

The architects for the project are Ahrends Burton & Koralek with

Ove Arup & Partners as structural, civil, mechanical and electrical engineers.

3.0 The architectural brief

After 20 years of engine production at Shotts, the original north-light factory (clear headroom of 3.6 m) had been expanded in 1970 by the addition of a two-bay standard,lightweight, steel-framed building with clear headroom of 5.5m. The plant was by now inadequate to service Cummins' planned expansion in production.

Apart from lack of space, the main disadvantages of the existing building were:

- limited headroom in the north-light factory
- layout restrictions
- no production loadcarrying capacity in the roof structures

There was also a desire to provide a high standard of ventilation in any new development.

All construction activity was to be organised such that engine production at the plant could continue uninterrupted throughout the development period.

The main production activities were categorised into:

- receiving
- machining
- assembly, followed by testing
- final assembly and shipping

These were seen as a linear process; raw materials and parts arrive, are checked, machined or stored, assembled, tested and shipped, which requires a series of production areas planned to terminate with the existing test cell complex (see Fig 1)

The major new manufacturing areas (receiving, machining, and assembly) required new buildings (around 17 000 sq m in total), with the particular requirements of a 5.5 m clear headroom and prescribed lifting capacity at any point on the roof structure.

4.0 Development of the design

4.1 Choice of structural grid

The best column spacing for a factory is generally the widest one that still leads to an acceptable price for the horizontal structure.

The 'acceptable price' was 50-60 kg/sq m for the steel structure, with 41 per bay as an approximate crane capacity target, early trial designs suggesting 15m as the likely maximum span. Cummins' factory processes were quite 'linear' and so suited well enough to a rectangular column grid with frequent columns 'along' the process line, but a square grid was favoured because of the need for adaptation to possible future layouts both for Cummins and for the Scottish Development Agency, who were concerned for the building's usefulness beyond the needs of the one lessee.

15 m x 15 m column grids were the norm for a 'general

purpose' factory with a 1-t roof craneage capacity.

A primary/secondary system of roof beams was preferred rather than any attempt at a more homogeneous grid, such as a space frame, because the bulky services runs were expected to be strongly directional in their space requirements and this would need reflection in the structure. Secondary beams were placed four per bay rather than three or five per bay on the basis of total steel weight comparisons for a typical bay including beams, purlins, and crane rails.

The decision to offset the secondary beams, so they bypassed rather than intersected the columns, followed from the choice of 'pin-ended' columns. With no moment connection to the roof, the columns neede only a single point of attachment; to underline this simple relationship truss and beam elements were set back from the column so that the pin became a visual focus. Or that was our hope. The effect was achieved only with much deliberation over the vulnerability of pin-ended columns to accidental damage

4.2 Structure/Services integration

The structural design involved seeking solutions based on the considerable 'servicing' requirements of the roof as well as its structural requirements. Given that process machinery was to be grouped along column lines to keep aisles free of both structure and plant, it was logical to concentrate primary runs for services along column lines as well. Two very different roof profiles that offer this

feature are shown in figs 3(a) and 4(a). The greater depth over the columns in the 'process direction' suggested it as the direction of primary structure, supporting secondary beams or trusses whose shallower depth provided a zone for lateral passage of the shallower secondary services runs.

The 'flat soffit schemes' (Fig. 3) offered steel structure easily accessible for monorail and light gantry crane attachement and so were favoured by the client, although the idea of a crane rail support system divorced from the roof and installed only where needed (Fig 4{b}) offered savings in steel and held sway for a while. Alternative handling methods (Fig 4{d}) were in favour only briefly, and honest assessment of deep crane rail suspension rigs (Fig 4{e}) led to steel weight increases, as well as loss of the visual and planning discipline that the architects were pursuing vigorously.

The roof in Fig 4(d) held appeal for the quantity surveyor and was used as a cost reference, though the building volume savings, and hence head load savings,inherent in the contoured cladding profiles of Fig 3 also had a discernible effect on cost. Fig 4(d) offered convenient access to ducts, pipes, and busbars, but this very lack of containment of the service militated against the desire for visual order in the roof space. No advantages attached to the intermittent zones of increased headroom in Fig 4 in the client's eyes; a uniform 5.5 m clearance was sufficient. No scheme contained an outstanding rooflight solution and some

of those considered are shown hatched on Figs 3 and 4.

5.0 The chosen roof structure

The scheme shown in Figs 5 and 6 finally emerged as the clear favourite. Primary elements are triangular trusses 2.0 m ddep, of fully welded CHS sections (maximum 200 mm). Secondaries are 686 mm series castellated beams, chosen in preference to UB sections for stiffness and because they neatly contain the 650 mm zone neede for passage of secondary service runs - especially roof drainage pipework, and lights. The beams are discontinuous where they pass beneath the trusses, and are supported there and at the attachment points of 48 mm inclined tierods that reach down from the truss top chords. The tierods have functions beyond supporting the beams. They define the trapezium zone for passage of primary services; they stabilise the top, compression, chords of the truss; in areas of high wind suction they work as struts in compression; and during erection, their threaded ends allowed control over beam and truss line and level.

Arrangement of the primary service run (ventilation duct, dust control ducts, compressed air, stream, busbars, sprinklers, water) determined the generous 2.0 m primary truss depth, which led in turn, to a low truss weight (2600 kg) compared with trusses of shallower depths. Likewise, the 686 mm depth of castellated beams, also dictated by space requirements for passage of services, led to stiffness and cost advantages ove shallower UB and truss alternatives. Some early criticism of the attempt to match structure and services to closely as being pedantic, now seemed ill-founded. Admittedly, though, the space

requirements of a 1000 mm ventilation duct and a 900 mm suction duct for swarf removal, were usual controlling factors.

Maximum span of the secondary beams is a low 5.0 m and they are continuous over three such spans, so the 'cable stayed' arrangement provides very generous support to the loaded surface. Indeed, for that part of the roof load that is dead load and so constant for each bay the tierods provide closely-spaced, balanced, unyielding support to the beams, and the structure is remarkably efficient. That is perhaps less true of its response to non-uniform live load such as snow load and crane loads. Live load in one bay only causes the tierod supports to 'yield' by forcing rotations onto the adjacent trusses. Tension across the tierod 'haunch' then activates the bending stiffness of beams in adjacent bays, which assist in receiving the original load. Even for live load, then, the system is considerably superior to beams of simple 15.0 m span.

Stability

As a prerequisite to design discussions about lateral stability systems of the 8.5 m high steel frame, the full spectrum of possibilities was sized and costed. Contraflexture in the columns (harder) under sway forces was placed below ground, in the roof, and at numerous points in-between. For fully braced solutions, various spacings were tried for vertical braced bays, and various locations for horizontal roof plane trussing. Steel weight differences between the alternatives were generally less than 10% of total steel weight, so there was scope for applying to the choice criteria less quantifiable than cost; the architects' preference among the offerings is shown in Fig 5 -

wall posts and raking props combined into tetrahedra stepping rhythmically along the facade and neatly fixing each line of beams individually. Columns could then be pinned north-south. For stability east-west, column lines were linked across to the relatively massive plantroom support frames, and propping forces considered to travel via the top booms of the primary trusses.

In steel weight terms, the, stability was achieved very cheaply. What may have escaped the quantity surveyor in confirming this, though, was the 'Pandora's box' of wall cladding and glazing solutions that the tetrahedral geometry opened up. The architects and engineers were quick to realise the aesthetic opportunities inherent in cladding the propped wall. The design that emerged (Figs 8 and 9) is a lively departure from a bland, vertical wall plane and has become something of a hallmark for the building.

To be accurate, the inspirational origin of the external props was not at all esoteric. During consideration of column moments as the means of stability, it was noticed that the frame in one part of the project would only be as wide as two column lines during one phase of construction, so those columns needed to be inordinately big. The expedient of temporary external propping was adopted, and was soon found to hold promise for the rest of the frame as well!

6.0 Influence of Architectural expression:

'Structural legibility' v. 'redundancy'

It became clear early in the design that the architects wanted

the structure not just to fulfill its several functions but to be seen to fulfill them as well. A question asked of a structural element was 'what is its primary structural purpose and how can that most clearly be expressed?' Thus the inclined hangers supporting the castellated beams became pure tierods with threaded ends, where they could have supplied support to an intermediate purlin and become bending members as well. Thus also the move toward pin-ended columns was welcomed. The columns were expressed as pristine tubular members with then sole function of propping emphasised by waisted pins top and bottom, in full view. (See Figs 10 and 11)

More subtly, the inclined facade props, located proud of the inclined wall cladding to express better their structural function to outside observers, generated large biaxial moments on the castellated beams under lateral wind load (Fig 13). The moments arose from the large eccentricity between external prop and internal wallpost - an eccentricity created when the prop and post were forced apart at their top joint to allow the cladding envelope to pass between from its normal position outside the structure, on the roof, to a position inside the structure, down the wall. Given the choice of receiving the weak-axis component of beam moment thus created by augmenting the beam flanges or by tying the tops of the external props longitudinally (tierod 'T' in Fig 13), the latter was selected as the 'purer' solution, and another set of tierods came into existence.

At many levels, then, choices were made that 'clarified' the functions of members and their joints. The eventual consequence,

of course, was a highly articulated structure of low redundancy. Whether such a structure is likely to be more or less edifying to its users than a more visually complex one (where, for instance, columns are generously tied into the horizontal framing and so resist wind as well as holding up the roof), is probably a less important question than whether such an attitude to structural design is consistent with other design attitudes throughout the building; whether it is becoming a contrived response to a simple problem; and whether it is becoming too expensive.

The practical consequence of low redundancy, of course, is the danger of progressive collapse. The frame was checked for the possible removal of various tierods and external wall props, and found still to be safe. Columns, the most vulnerable elements of the frame, were designed specially for impact loading. (see 'Impact on columns')

7.0 Influence of roof-mounted plant on structural design

As planning progressed for the layout of plantrooms and services distribution, the roof structure was periodically reassessed by the architect as a possible repository for plant. The five 11 kV substations, for instance, were nearly roof-mounted. The only major items finally to remain in the roof were 16 'dust collection units', beneath the 'blister' housings in Fig 2, which filtered air laden with particles from the machining processes. Normally, these units weighed 5 t, but failure of a discharge conveyor could lead to 'clogging' of any unit and a weight increase to 16 t.

There were two ways of receiving such heavy and variable loads - either by trying to dissipate their effect over as large a spread of roof beams as possible or by trying to localise their effect by providing just a few lines of specially strengthened beams. Partly because the location of these dust units was likely to be the subject of some change during design development, the latter course was chosen.

To avoid spreading large perturbations from these loads, however, a special precaution was necessary. Dust units were assumed, pessimistically, to occur in alternate bays (analagous to 'snow' in Fig 18), and with the further assumption of one unit 'clogged', alternate sagging and hogging in the bays of castellated beams caused, in turn, twisting of the primary trusses supporting these beams.

To avoid this torsion transferring the large moments to other lines of beams across the roof, the trusses had to be rendered very flexible in torsion. This was achieved easily by failing to triangulate the top surface of each truss. It was then necessary only to check the magnitude of potential torsional strains and the bending stresses they produced in this top surface of truss members.

There was an unfortunate corollary, though. Plans to couple adjacent lines of castellated beams by truss torsion so that a crane load on any one beam would activate a large number of nearby beams in a kind of grid action, had to be dropped. The obvious lesson was to try to avoid heavy local loads in a continuous lightweight roof.

8.0 Beam design

The castellated beams were modelled as 'Vierendeel trusses' for detailed analysis.

The compression force generated in the beams by tierod tension was founded by computer analysis to exist almost entirely in the top chord of these 'trusses'. Thus a typical beam was sized on the basis of limiting the maximum total longitudinal stress at the top and bottom edges of the 'T'-section constituting this top chord, considering overall beam bending, bending due to shear, and compression. Bottom chords attracted high compressive stresses when patterned crane and live load in the macro model put alternate beam bays entirely in negative bending, but this was rarely critical for design.

The effective length of the top chord for compressive stresses due to beam bending and pure compression was the distance betwen horizontal longitudinal tierods linking adjacent beams ('R' in Fig 18). Another check concerned nett section stresses in top and bottom chords of this 'Vierendeel truss' model, on the assumption of two 22 mm bolt holes in one beam flange.

None of the structural joint details or services attachment details called for such holes in potentially highly stressed areas, but it seemed a wise precaution to allow for them. In fact, none of the roof mounted equipment so far installed by the client has required flange bolting; clamping devices have been used in every case.

Secondary effects included compression along beam lines due to

- the horizontal restoring couple required for equilibrium of the pin-ended columns during a north-south thermal contraction of the roof

- stabilising forces required for compression in both columns and top chords of primary trusses.

- braking of cranes

There were no reasons for using steel stronger than grade 43. the 686 mm castellated beams provided the necessary depth for secondary services runs and were found simultaneously to attract stresses suitable for grade 43. Likewise, truss members were tubes of suitably small diameter, and columns and tierods that pleased the architect were also working at convenient stresses. Deflection checks on the roof considering severe crane and snow layouts gave acceptable results considering drainage, crane rail gradients, and distortion of sheeting and glazing and services runs, while stronger steel with higher stresses and strains could have led to serviceability problems.

9.0 Impact on columns

The decision to articulate the structure by incorporating pins at the top and the bottom of the steel column makes the column more than usually susceptible to accidental damage. Because of the nature of the building, with forklift trucks being used to transport materials, the probability of collision is quite high and cannot be discounted. The possibility of using a form of protection separate from the column (eg. bollards) was

considered, but was rejected because of the space it would occupy - also the requirement for flexibility in building use would dictate that the protection would have to be installed about all columns, which would cause even more disruption in the planning.

It was therefore concluded that the columns should be designed to be able to accommodate impact loading. The impact to be designed for was a range of forklift trucks impacting betwen 0.3 m and 1 m above floor level. The truck was assumed to have a kinetic energy of 15 kj . There are two mechanisms by which this energy could be dissipated, apart from distortion of the column. The first was distortion of the truck, but as this would be very difficult to quantify, it was not included; the second is rebounding of the truck, which is dependent on the equivalent mass of the column being greater than that of the truck and, as this was not obvious, it was not included. The full 15 kj was therefore assumed as having to be dissipated by deformation of the column.

The diameter of the steel column was 324 mm, with 10 mm thick walls in grade 43 C steel to accommodate the loads from the roof. It was not considered appropriate to change this, nor did there seem to be much advantage in doing so. To prevent local crushing, it wad decided that the columns would, where necessary, be filled with grout.

Schemes considered.

The following were considered:

1) Full height steel columns. The first point considered was

whether the bottom pin of the steel column could remain at ground level. ie. whether the steel column could take the impact.

2) Column restraint at floor level. For the impact at a height of 1 m, it was found that, although the impact itself would not cause collapse of nominally loaded column, the column could be left with a kink of approximately 100 mm and in this state it could not support the full vertical load (Fig 20{a}).

For the impact at 0.3 m, it was found that the shear necessary to be tranferred by the pin at the bottom of the column was too high. (Fig 20{b})

3) Column on concrete sub column, which extends from ground level down to top of foundation. Two heights of plinth were considered, 1.2 m and 3.5 m, both 450 mm square. In the case of the 1.2 m high plinth, it was found that the steel column could yield before the concrete column and so acted as the case above. If the concrete did yield, the ratio of the plastic to elastic deformation required was too high at approximately 35 (Fig 20{c}).

In the case of the 3.5 m high plinth, the total movement of 220 mm was felt to be unacceptable (Fig 20{d}).

The estimated deflection of the 3.5 m high plinth decreased to approximately 80 mm if backfill was taken into account,

but it was generally felt that the depth of excavation associated with this plinth was too great.

It was therefore considered necessary to lift the bottom pin of the steel column on top of a concrete plinth and consider the impact on the concrete plinth.

4) Column on concrete plinth (Fig 20(e)). One effect that was not considered with the plinths below ground, which became more important with the plinths above ground, was the restraint that could be provided by the ground slab. The height of the plinth was set at 1 m above ground floor and to extend 1.2 m below ground to the top of the foundation, to give an acceptable amount of excavation.

It was found that, if the ground slab did not restrain the plinth, for the minimum plastic moments in the concrete the deflection was too great (approximatley 130 mm). However, if the ground slab restrained totally the plinth for the maximum plastic moment in the concrete plinth, the shears were excessive.

It was therefore decided to limit the gap at ground level to approximately 50 mm, such that where the maximum plastic moments are considered a plastic hinge is formed only at the foundation while, for the assumption of minimum plastic moment capacity, a plastic hinge is formed first at the foundation and then at ground level.

In order to limit the shear in the plinth due to impact 0.3

m above ground level, the above ground level section of the plinth was made weaker than the part below ground.

Solution adopted As the design was refined, some modifications were made to the solution described above. The height of the plinth was increased to 1.2 m and was made circular, the diameter being approximately the diagonal of the 450 mm square column, i.e. 650 mm. It was considered necessary to design the pinned connection for the steel column to take account of an impact at the bottom of the steel column.

Although the forklift truck would not impact on the main part of the column, the bottom 3 m of the column was filled with grout to prevent damage due to swinging loads suspended from the overhead cranes.

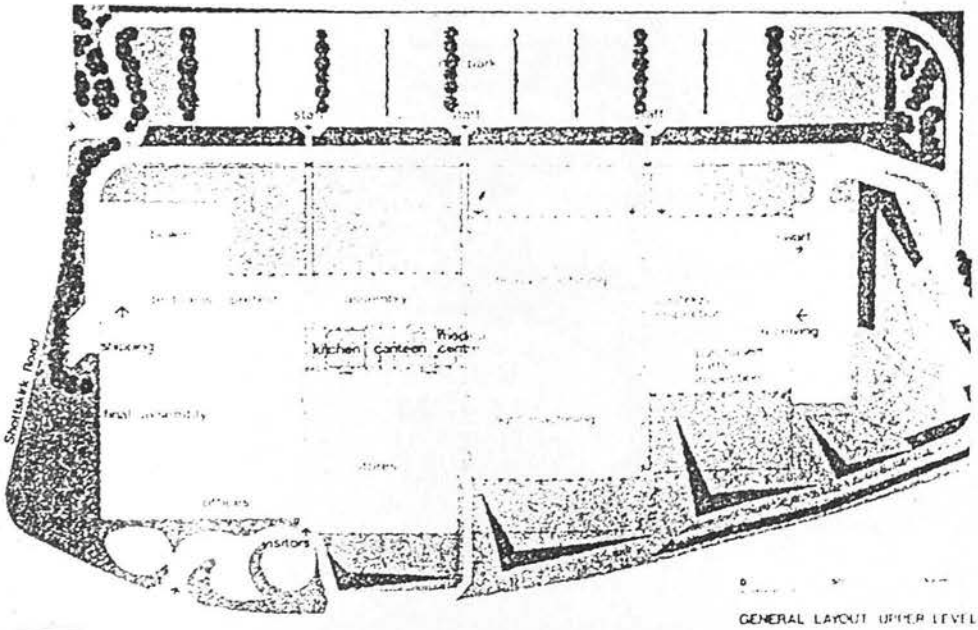
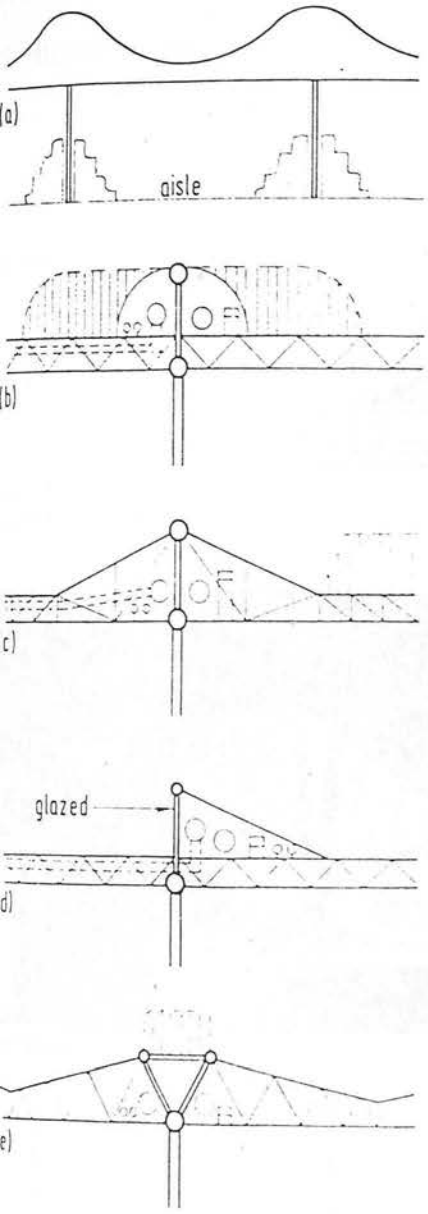

Shottskirk Road
kitchen
canteen
GENERAL LAYOUT UPPER LEVEL

Fig 1



offs schemes considered

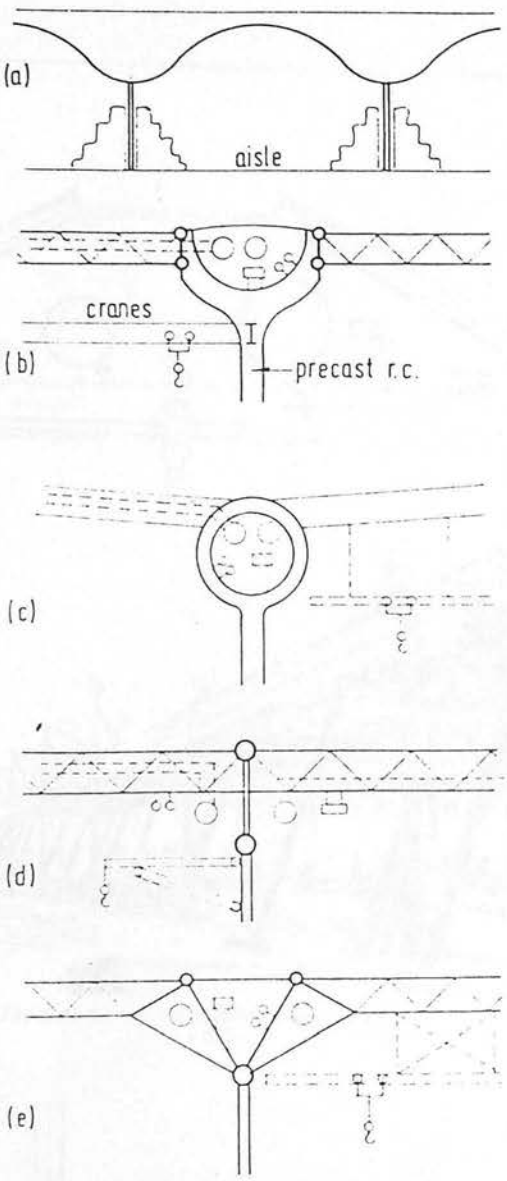
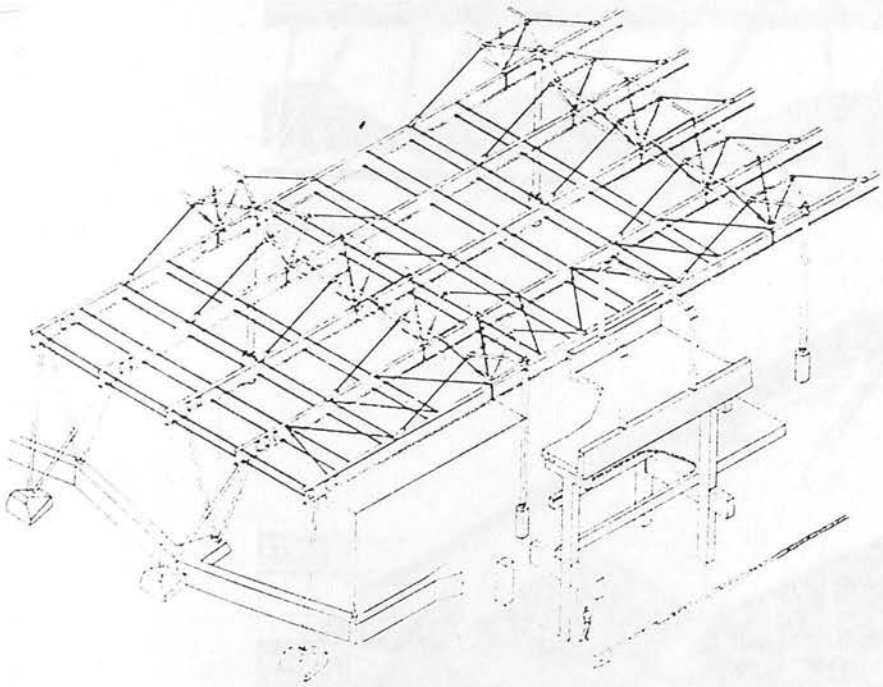


Fig 4. Flat roof schemes considered



of part structure

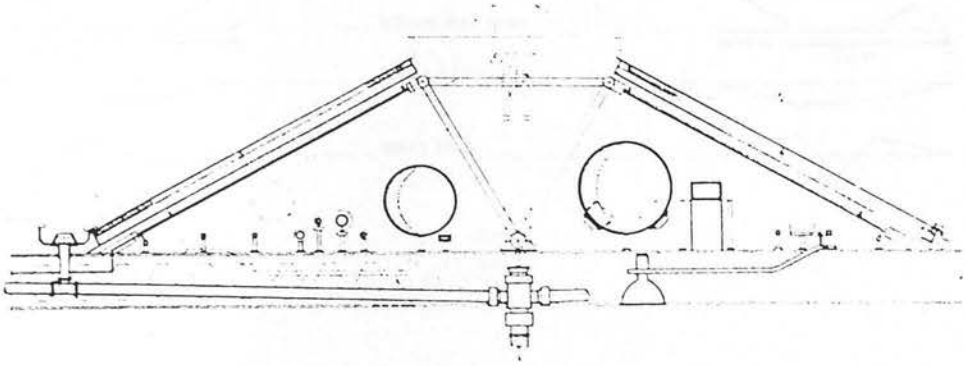


Fig 6. Cross-section through roof, showing service routing

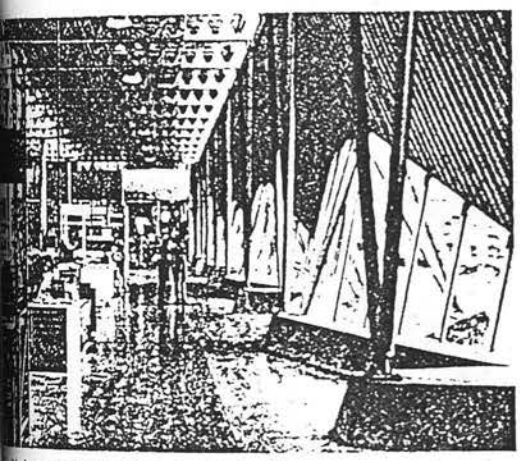


Fig 7. Internal view showing articulated wall

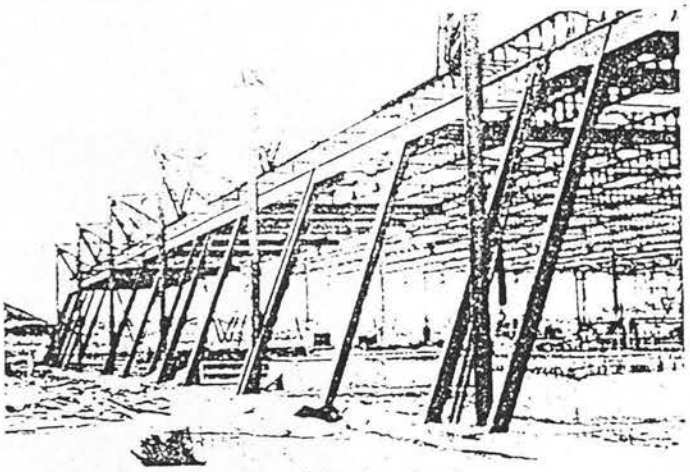
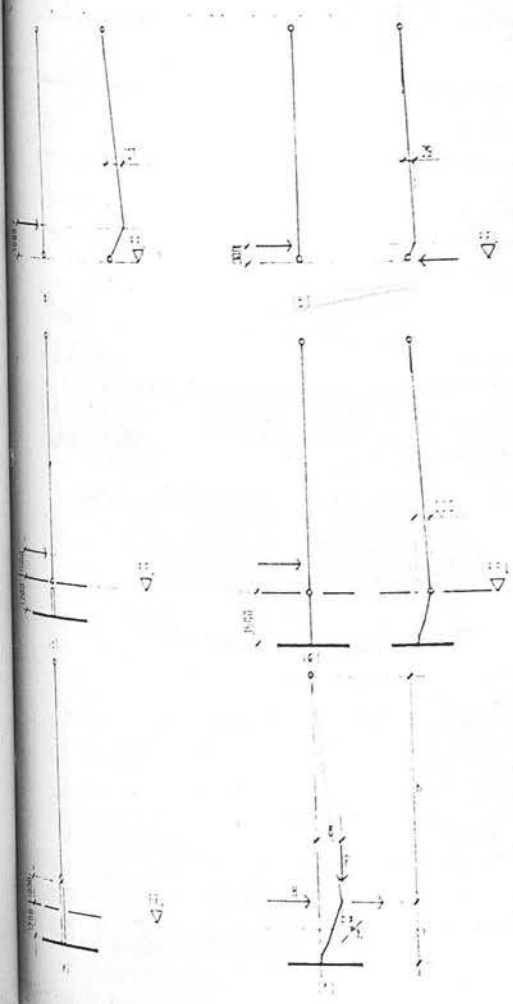


Fig 7. The structure at the receiving door facade



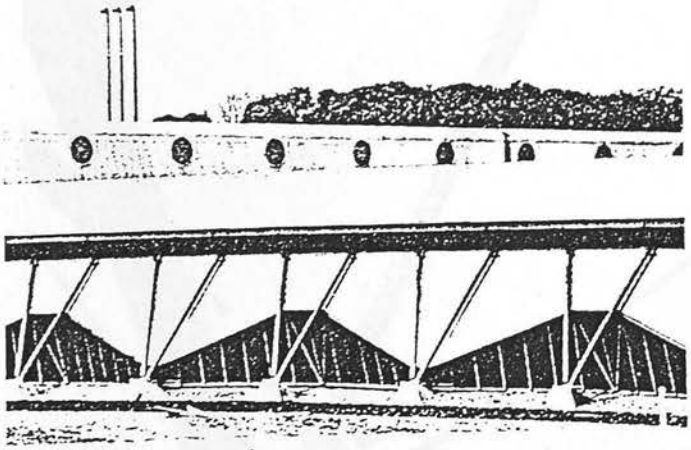
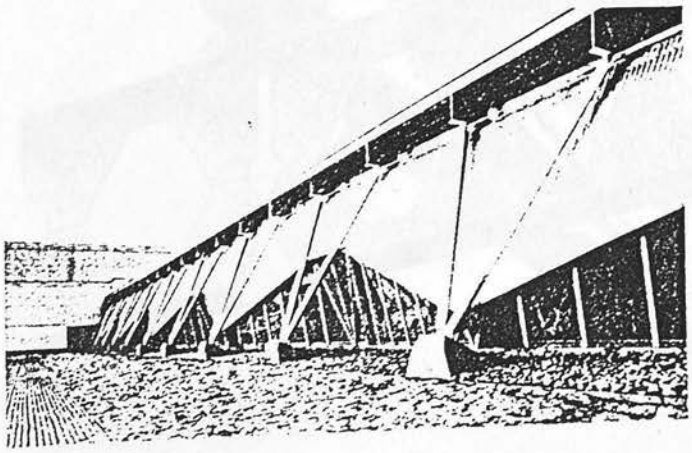


Fig 8. Raking wall elevations



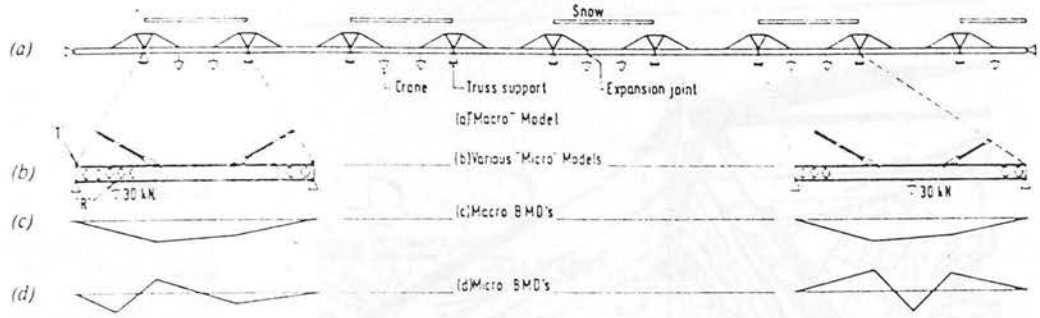


Fig 18. Roof loading and analysis

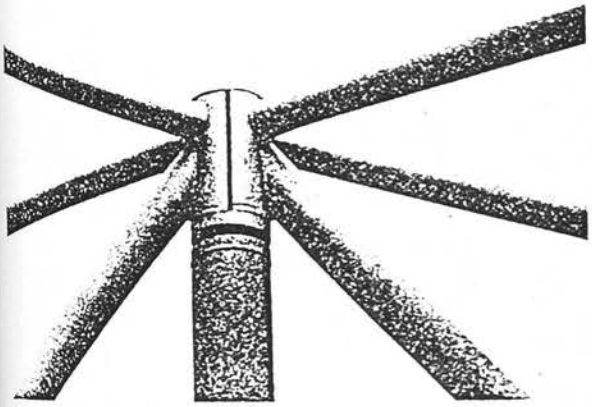


Fig 10. Truss-to-column joint

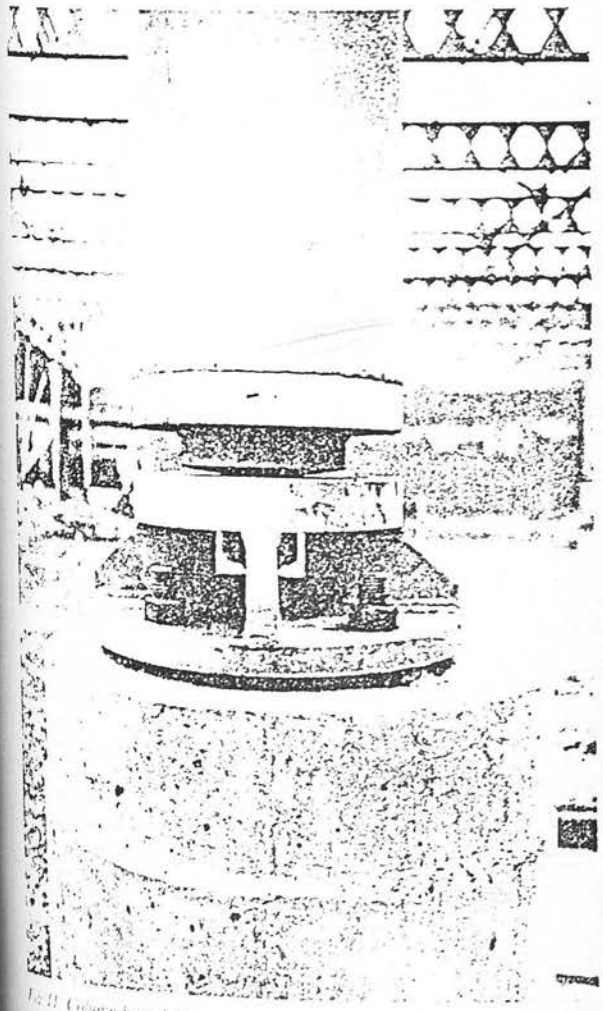


Fig 11. Column base detail

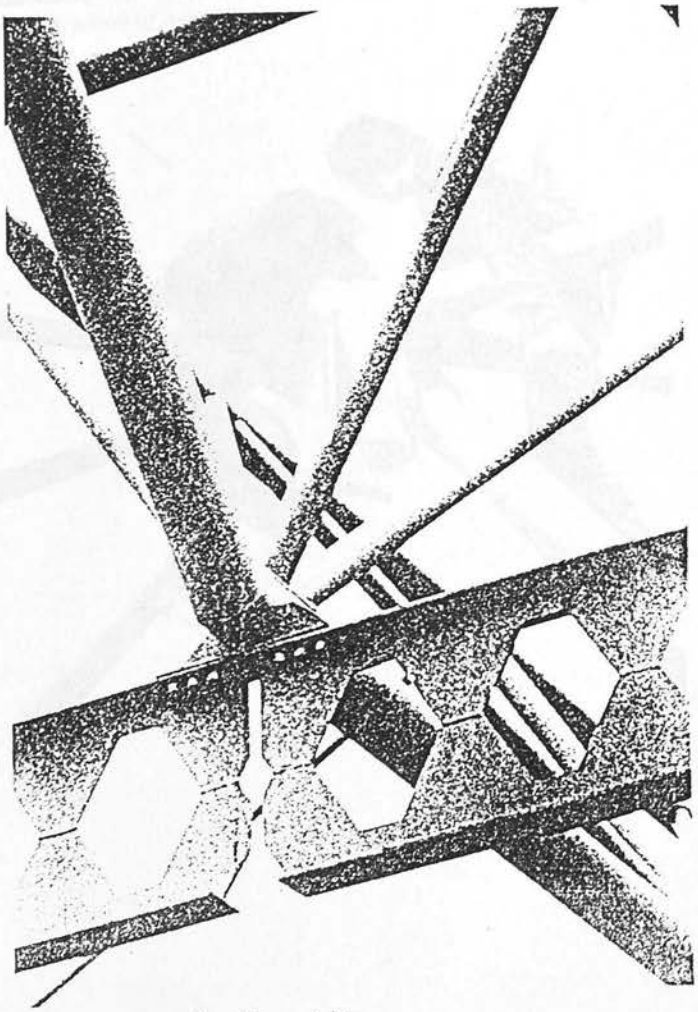
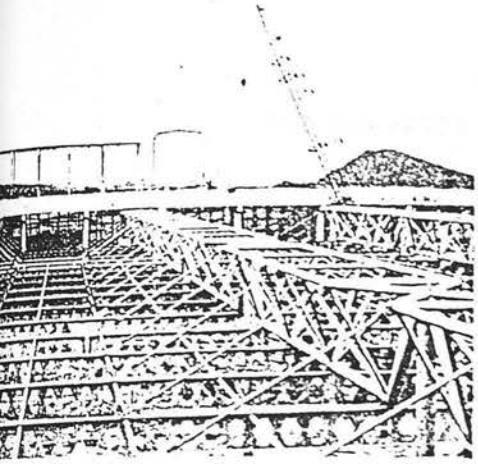


Fig 12. Truss castellated beam joint



completed skeleton roof without purlins

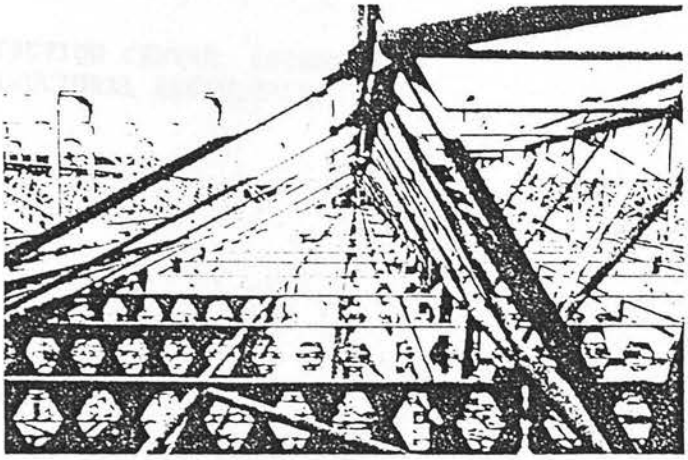


Fig 14. View through service void, showing hangers

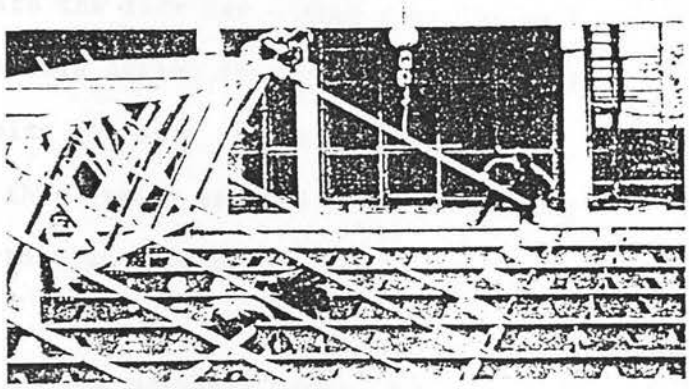
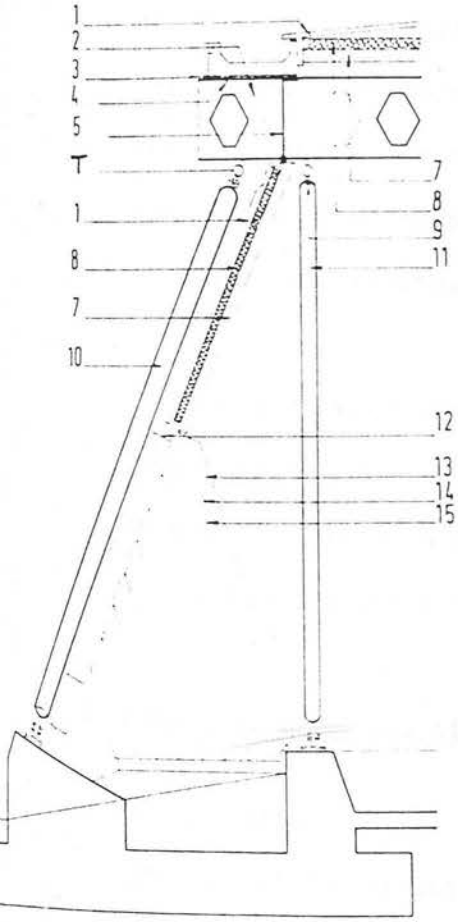


Fig 15. Erection of hangers



ofiled sheeting
5 × 152 castellated beam
× 89 steel channel
mm steel sheet
mm toughened green anti-sun glass in neoprene gaskets
rod
ofiled perforated steel decking
0 mm insulation
4 od steel tie
8 od CHS inclined prop
7 od CHS vertical prop
0 × 200 steel angle as gutter and cladding support
mm steel mullion
ent glazing bar
mm toughened green anti-sun glass

Typical section through external wall

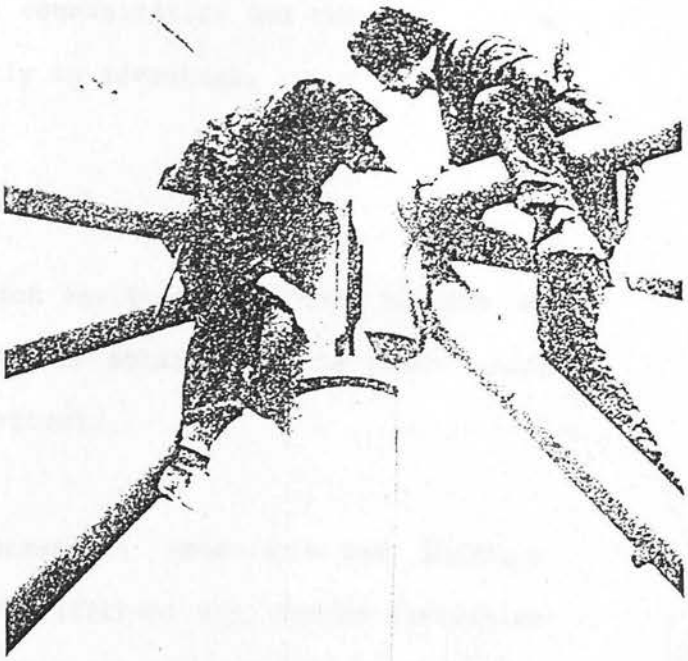


Fig 16. Erection of [illegible]

CASE STUDY No 15

RENAULT PARTS DISTRIBUTION CENTRE, SWINDON: THE CIVIL AND STRUCTURAL ENGINEERING

Architect: Foster Associates
Extracts from a paper by
Martin Manning, OAP
OAP Journal Sept 1983

1.0 Introduction

Renault, Swindon, can perhaps be seen as a member of a family of steel roof structures on which the firm has worked over the past few years. Its form and details derive from considerations of structural behaviour to minimize the size of the main components. Some of the other members of this family are the Cummins Factory at Shotts, and Fleetguard Factory at Quimper.

This particular building forms the National Parts Distribution Centre for Renault UK and also houses their regional office. The proximity of the road and rail communication and the port access to the west and south is clearly an advantage.

2.0 Commercial advantage of the proposal

The proportion of the site which was to be developed was 50% as approved in the planning permission obtained by the client prior to Foster's appointment as architect.

By approaching the general arrangement, appearance and location of the building on the site in a different way, Foster Associates were able to convince the local planners that 67% of the site might quite reasonably be developed and the planning permission

was amended accordingly.

3.0 The general arrangement

The brief required 20,000 sq m of warehouse, a training school, a showroom and an office, totalling in all some 25,000 sq m. A principally single storey building with a footprint of that size would clearly imply a substantial earthworks contract on any site.

The site was a green field to the west of Swindon with a fall across it of about 5 m.

After a series of cut and fill studies it was decided to place the building at a level which would require approximately 2 m cut and 3 m fill and to re-use the excavated clay as fill material underneath the ground-bearing slab.

Considerations were also made of the appropriate module size for a multi-use building of this kind. The sizes of the stored pieces in the car industry do not lend themselves to automated storage systems. Such warehouses operate on the principle of a staffed supermarket and require the facility to move racking and storage about throughout the life of the building. With the requirements of the showroom and training school it became apparent that a module size of the order of 24 sq m with an 8 m internal height would be appropriate for such a development.

The shape finally chosen for the building and its position on site allow up to 50% expansion of the warehouse with comparable

expansion of the support accommodation.

The ubiquitousness of the enclosure system followed the lead set by a previous building of Norman Foster's in Swindon, Reliance Controls, and in a sense this was the starting point for the whole envelope.

4.0 Development of the Structure

4.1 The Form

The original proposal was a sketch made by the late Ken Anthony. It showed a single mast with radiating beams hung from its top. In this way the dead load of the roof either side of the mast could be balanced, and bending continuity of the beam system provided, over the top of the column and outside the building envelope. The origin for this was no doubt the experience Ken had had working with Norman on the development of the Headquarters for the Hongkong and Shanghai Banking Corporation.

The umbrellas started as separate independent structures and the system was always conceived as being two-directional rather than one. (Fig 1)

The building was always going to be very large. To provide bracing to the roof structure would have probably implied expansion joints and bracing positioned internally.

It was decided to develop the design as a two-directional

portal frame. The structure was to be composed, principally for reasons of appearance, of a continuous bending column and continuous bending beams pinned to those columns, connected and stiffened by members capable of taking tension only. For such a structure, variations in imposed conditions, whether they be loads or strains, dictate the elements available to resist those imposed conditions.

Uniform downward loads stress the tension members. Sway forces or wind uplift forces destress some of the tension members. The resistance of the structure to horizontal disturbance is then small. The first aim was to define the members permanently available. The structure was made sufficiently stiff so that modes of failure due to instability would not interfere with yield.

To achieve this the ties adjacent to the masts were prestressed to provide not only a moment connection between the beams and columns but also to strengthen and stiffen the columns themselves. The main span tension hangers are principally activated by downward load. (Fig 2A, 2B)

4.2 Effects of wind

Checks were carried out on three of the dynamic features of cable-stayed structures:

1) do the ties shed eddies so that they vibrate cross-wind?

2) Do they gallop downwind?

3) Does the roof 'snatch' under wind load?

ie. consider the case when wind is blowing and the roof has been lifted by a large eddy and some of the tension members are inoperative. The eddy passes on; the beam stiffness is now inadequate to support the roof; it starts to deflect downwards. At some point in its passage the tension members will become re-stressed. The stiffness of the roof in its downward passage under gravity loadings will increase. It was necessary to demonstrate that the effect of overstress on the ties due to this change in stiffness was not great.

5.0 Detail design and fabrication

5.1 Many of the details of the structure are unusual. The shape of the beams reflects the bending moment envelope under all load conditions. If a beam is to be tapered in this way then the web is cut at an angle, the section reversed and then rewelded. In such a roof beam shears are not large and the web rewelding need not be continuous. Between the welds circular holes have been cut out for architectural reasons.

The tie members are Grade 50C where they can possibly be destressed, and Macalloy bars where they are prestressed around the column.

To achieve uniformity of the connection between the ties and column for the two types of tie, a mechanical coupling appeared the only option. Many of these couplers would be external and thus a smooth, well-sealed profile would improve durability. It was decided to cast the pieces in

either iron or steel. Iron was chosen on grounds of cost.

5.2 Construction modification on design

The castings were connected to the columns by single pins through vertical plates welded to the wall of the column. In the tender design these plates passed through the column. All stiffening to the section was provided internally. Because of the requirements of the programme and the difficulty of obtaining the tube, the management contractor had decided that the columns had to be ordered before the steelwork sub-contractor was appointed if the project was to proceed to programme. As a result of the tender it became apparent that the cost of the details within the column was high and that all stiffening to the column should be external rather than internal. the design was changed so that the vertical plates were restrained by annular rings placed around the column. Such a configuration of course is exactly that most like to induce lamellar tearing in the wall of the column. In order to assess the sub-contractor's welding procedures, samples taken from the columns to determine the through thickness properties of the material were tested and the results were compared with published information to establish the likelihood of lamellar tearing taking place. Even though many of the lengths of tube had been made from the same billet, the values of the short transverse reduction of area (STRA) tests were all different. Thus, the distribution of non-metallic inclusions within the walls of the column was almost impossible to predict and lengths of tube could not

therefore be selected to eliminate the defect. However the sub-contractor clearly had a problem in fitting annular rings onto tubular columns. The 457 mm diameter columns varied by as much as 32 mm across orthogonal diameters. A full strenth butt weld between the wall of the column and a 40 mm thick annular ring would produce a huge weld where the gaps were greatest. To overcome the problems of both tearing and tolerance the sub-contractor suggested buttering the surface of the column with weld metal before fitting the annular ring to ensure a precise fit. All of the welds in these columns were ultrasonically checked and no lamellar tearing took place.

6.0 The erection

Once the first pieces of steel had arrived on site the first task was to assemble the prestressed elements; (ie. the column plus the four stub beams and the eight prestressed ties) and prestress them checking both load and position. The prestressing was carried out using Pilgrim nuts. This is an hydraulic jacking system devised in the shipping industry for tensioning bolts. the units were assembled to the same level of tolerance as we might require of a fabricated unit. They were then erected onto the bases.

The main grid line beams and the diagonal beams were assembled on the ground with their lower tension trusses only. They were lifted into position between the columns and the single pins inserted. The ties from the top of the column to the quarter span points on the beams were then attached to a strong back, set

to the exact straight line length and placed in position. The nuts on the ties within the beam were done up finger tight.

The strong backs on the tie were then removed. Under self-weight the ties would sag and pull the top of the column over, and the quarter span point on the beam, up. Adding purlins and internal bracing subsequently straightened the ties. When the steel was complete in a bay the loads in the ties around a column were all checked with a Pilgrim nut at the same time as the column was plumbed. (Fig 4, 5)

On Renault the angle of inclination of the tie, the weight of the tie, and the tension induced in it by the weight of the steelwork alone are such that the ties were sufficiently straightened so that their load extension characteristics were very nearly linear elastic. The weight of steel in the roof was similar to many other projects of this nature. Approximately 25,000 sq m of roof used 1400 tonnes of steel with some 27% being in the connections. Steel with such heavy details incurred some cost penalty with an average cost of 1150 pounds/tonne at September 1981 prices.

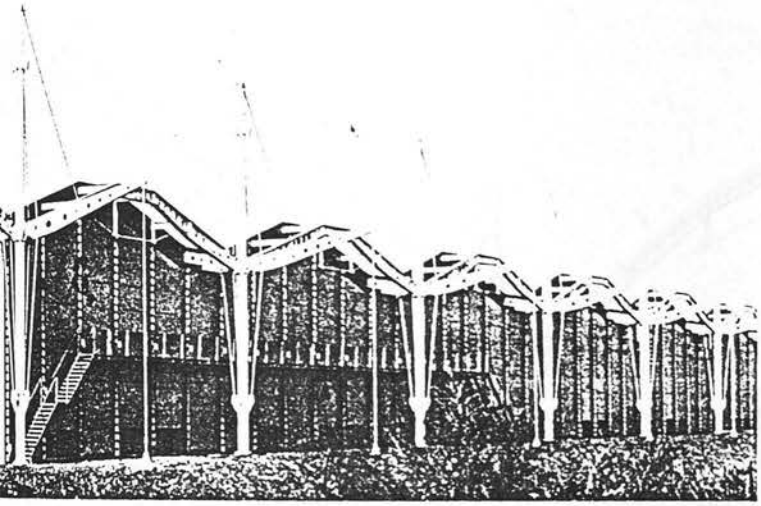
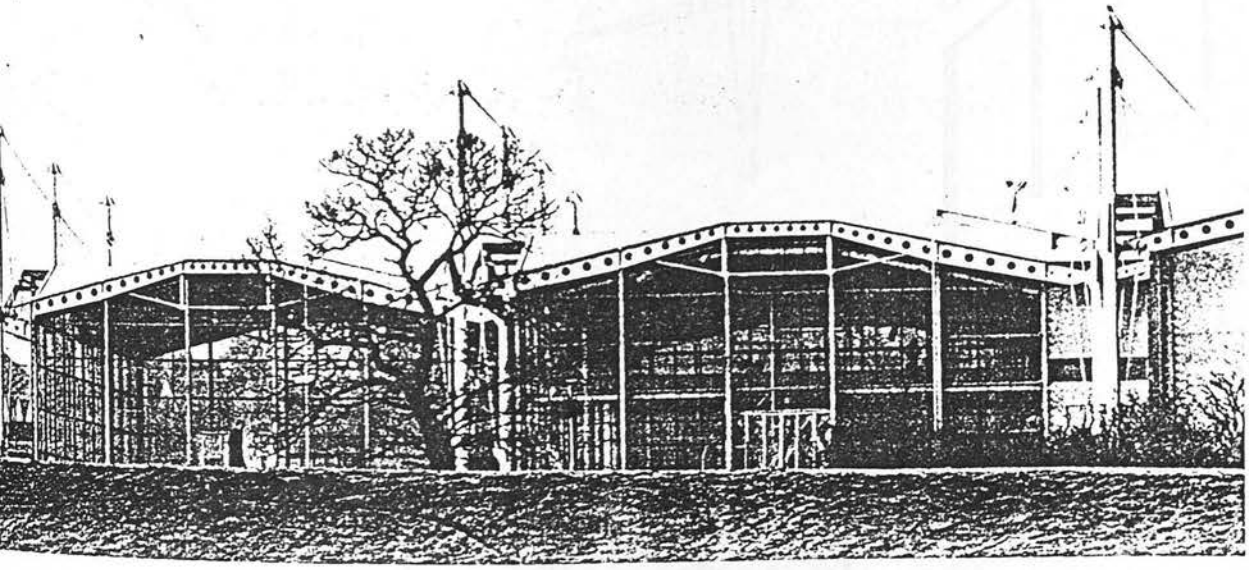


Fig. 1
The fire escape walkway and stairs which cantilever from the wall mullions allow the use of multilevel racking in the warehouse.

Fig. 2a
The prestressed ties at the mast stiffen the tube and provide the beam/column fixity; the outer ties reflect the overall bending moment envelope; the beam depth reflects the local bending moment diagram; the beam rise provides the inverted catenary to resist wind uplift.



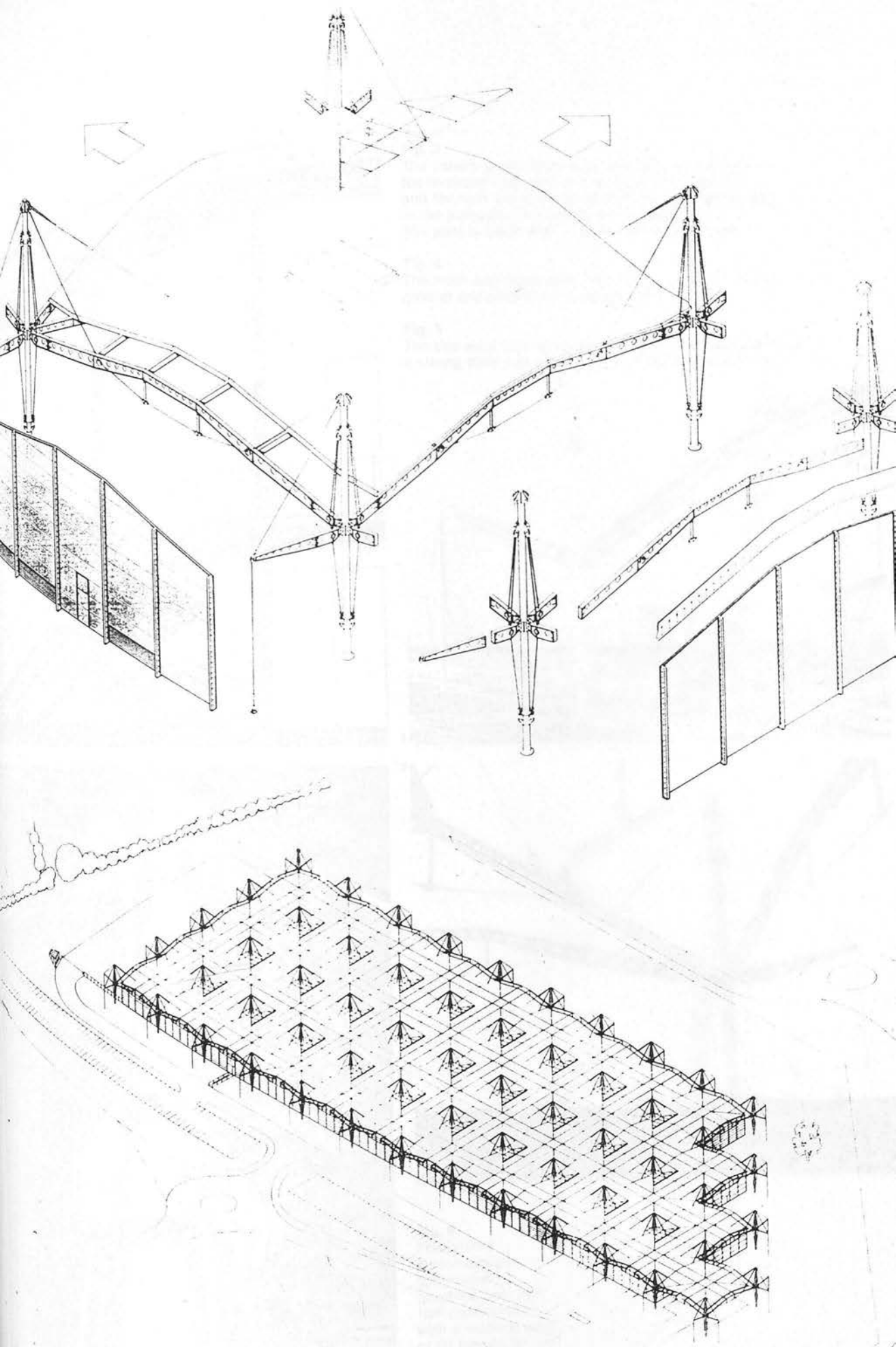


Fig 2b
Structural priniciples—the masts support trussed portals on grid and diagonals

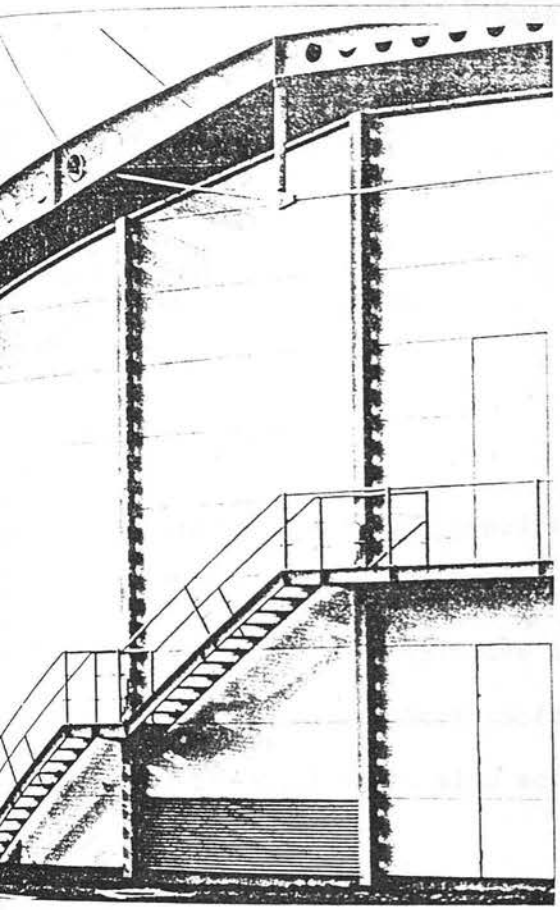
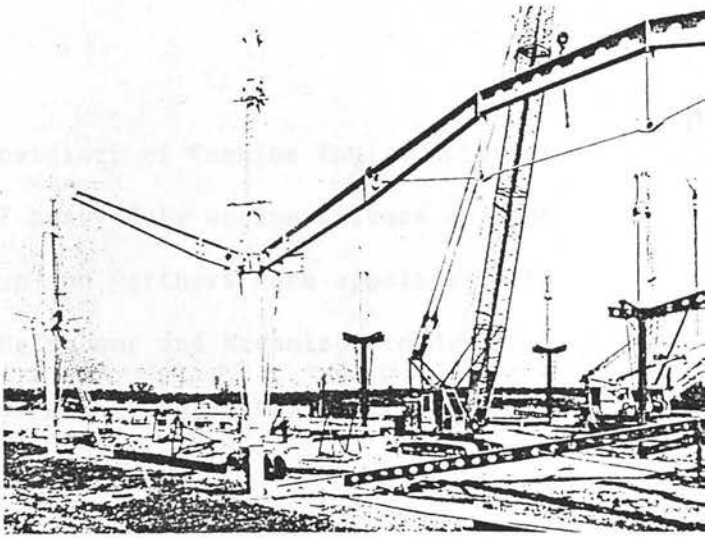
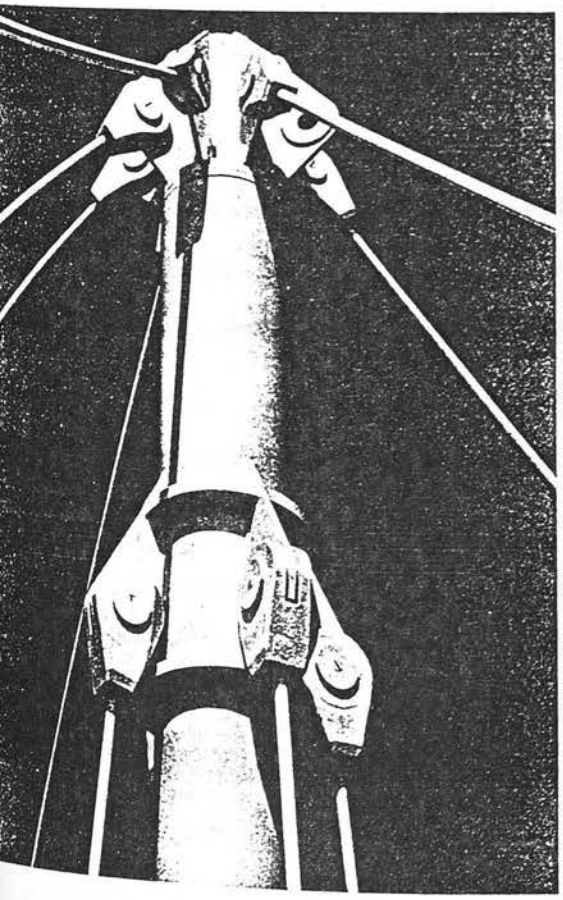


Fig. 3
The panels span horizontally between the mullions; the mullions span vertically between the slab and the roof; the roof moves vertically and horizontally in the plane of, and relative to, the wall.
The joint is made with a tensioned neoprene skirt.

Fig. 4
The mast and beam units were fully assembled on the ground and erected as complete units.

Fig. 5
The ties were erected to the precise straight line length on a strong back and subsequently set to load with a Pilgrim nut.





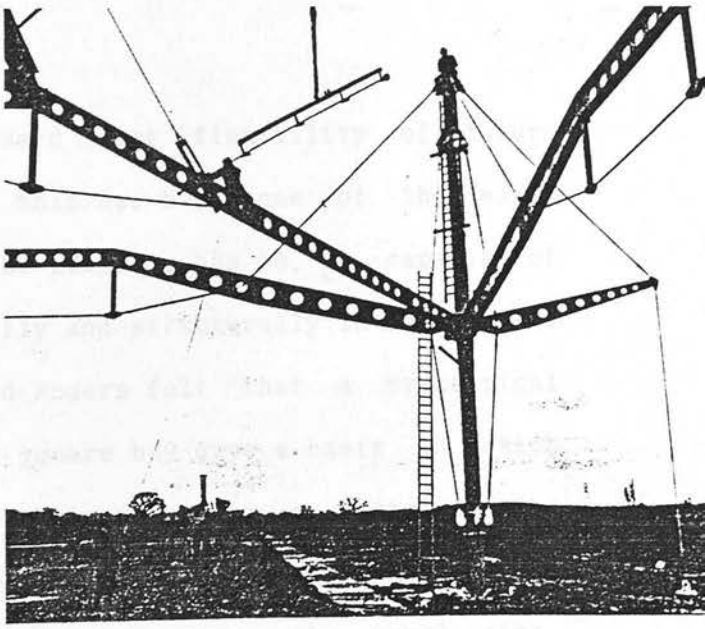


Fig. 6
Foam-filled silicon-sealed, spheroidal graphitic cast iron fork connectors with a working load of 60 tonnes for just under £100 each.

CASE STUDY No. 16

FLEETGUARD, QUIMPER

Extracts from a paper by
John Thornton

Architects:
Richard Rogers & Partners

1.0 Introduction

Fleetguard, a wholly owned subsidiary of Cummins Engine Company, is the largest manufacturer of heavy duty engine filters in the United States. In 1978 Ove Arup and Partners were appointed with Richard Rogers and Northcroft Neighbour and Nicholson to design a factory which would also act as the headquarters of Fleetguard, Europe.

2.0 Overall concept

Experience has shown Fleetguard that flexibility of future extension is essential and this has been one of the major concerns in the design. The building has to be capable of extension, both architecturally and structurally in small units on all four sides. Richard Rogers felt that a symmetrical external structure based on a square bay gave a basis on which expansion could take place while retaining a logic to the external appearance. It gave scale and grain and minimized the clad volume which reduced the visual impact on the rural site. He also wanted the structure to be as visually light as possible and this suggested a tensile structure.

In order to justify the increased complexity of such a structure, compared with a conventional structure, it was decided that the quantity of steel used should be less. This self-imposed criterion led, on a number of occasions, to the rejection of a technically possible solution and forced us to look for greater efficiency.

3.0 Roof structure

3.1 The building consists of an assembly of 18 m square bays. Initially there were to have been 28 bays arranged 7 x 4, but cost cutting reduced this to 25 which was an early justification of the design (Fig 2).

The roof, which is a composite panel system on steel beams, is suspended on a 6 m grid by tubular steel hangers from an arrangement of rods. The rods are attached to the tops of tubular steel columns which are arranged on the 18 m grid. The columns are 17.25 m tall and at the 9 m level, where they pass through the roof, they are connected to it (Figs 3 & 4).

Air-handling units and a cooling tower are suspended from the tops of the hangers and the roof must be capable of supporting 8 tonne ovens hung underneath.

3.2 Development of roof structure

3.2.1 The roof structure is the most important feature

of the building design and it is described by explaining its development.

Initially the roof was to be suspended from a horizontal array of tubes 1 m above, and this array, in turn, suspended by cables from the column heads (Fig 5). This emphasized the separation of the building enclosure from the tension/compression elements and ensured that the hangers always penetrated the roof skin at right angles, which made waterproofing easy. There was duplication of structure in this scheme which was uneconomic, and so the option of suspending the roof directly and exposing only the tension elements was studied (Fig 6). However, both the systems described above performed badly under out-of-balance loading between one bay and the next. The requirement to suspend did not help.

The system was highly interactive and there were number of factors involved; the problem was that when load was applied to the roof on one side of the column without being balanced by an equal load on the eother, the forces in the rods were unequal and the column was pulled over. This had the effect of lifting one set of beams and allowing the other to sink (Fig 6). This movement carried on until the bending forced in the beams and column were sufficient to balance the loads. The entire reason for the suspended roof was that

beams should only span 6 m and carry the weight of the roof alone. To make these beams then have to carry larger loads over effectively larger span was incorrect. Also, apart from the bending over the 6 m span and some unavoidable secondary bending, the intention of the structure was that forces should be carried in tension or compression and so substantial bending was contradictory. Apart from these philosophical objections there was another, more mundane.

Because the beam forces were largely controlled by deflections of the columns and cables, the more the sections were increased to carry the forces, the more forces they attracted. Eventually the system could be made to work but it was uneconomic and the sections look too bulky.

The solution to this problem lay in developing a system wwhereby the deflections of the structure in one bay were not transmitted into the adjacent bays. This was done by tying each column head back to the adjacent column at roof level. The force-inducing deflections were then almost entirely controlled by the axial stiffness of the cable system. As part of this arrangement it was also possible to arrange for wind uplift forces to be taken by the tension structure rather than by adding dead weight or increasing beam stiffness.

There are thus three load-carrying systems. One system carries downward loads, the second carried upward loads and the third carries out-of-balance loads (Fig 7). This extra complexity of structure was something that was to be avoided because of possible complications in assembly, but it was also recognised, that it was essential for economy and by careful planning and detailing there were no problems during construction.

One result of this system is that, because the tension elements are effectively continuous over the whole roof, it is not possible to incorporate an expansion joint. The structure has been designed to absorb the stresses due to thermal expansion of eight bays and, when the building is extended beyond this, a non-standard expansion bay will be introduced.

3.2.2 In restraining the perimeter columns, taking a rod directy from the column head down to the ground was considered. Horizontal anchorage forces, and movement of the anchorages had to be strictly controlled to avoid deflection of the roof. Unless the angle between the rod and the column was quite large the forces generated in the rod column and anchorage were large and also, with a small angle, small strains in the rod caused large deflections. On a sloping site the anchorages varied in distance from the column and the rod

lengths varied. Faced with these disadvantages, the rod was taken down to the anchorage vertically by passing it over a boom; and the penalty of horizontal forces transmitted through the structure had to be accepted. (Fig 4). The connections between the 18 m grid beams which carried substantial axial force and the columns, were originally to have been bolted. However, by using pinned connections, the large support movements were eliminated and the problem of detailing a connection to accommodate both an I-beam or a tubular boom, was resolved.

3.4 Although the system was originally conceived as cable-supported, rods were chosen when it was found that stiffness was more important than strength. The stresses were low enough to permit the use of rods rather than cables and the reduced axial stiffness of cable was a penalty. Rods are cheaper than cables and can be painted and re-painted, whereas the economic corrosion protection of cables has yet to be solved.

3.5 Construction considerations

3.5.1 Originally the structure was designed so that the centre lines of all members intersected. However, this meant that around each column, eight rods would penetrate the roof at an angle. By connecting the rods to the columns immediately above the roof and accepting a small amount of

bending in the column, this waterproofing problem was eliminated. The structure thus penetrates the roof on 6 m centres only and the penetrations consist of tubular elements. A conical shroud is welded to these and the waterproofing dressed up underneath. The number of structural penetrations is not significant compared with the number of services penetrations.

3.5.2 The structure is designed so that the bulk of it consists of elements which require very little fabrication and are joined simply. The single complex operation is on the column at roof level. At this point four beams and eight rods are connected to the column and all three elements may carry large loads. The fabrication involves notching a 9 m section of tube and inserting a spider assembly which projects through the notches to provide the connections. After this a second section of tube is welded on.

3.5.3 Site connections are either bolted or pinned and the major structural connections which have to be made in the air are pinned.

The structure is not pre-tensioned, only made taut. The minimum of adjustment necessary to achieve this is provided. The principle is that the position of the top of each hanger is defined

by the two rods coming down to it but that this position is not important. All that is then necessary is to provide adjustment to the rods connecting the hanger tops to each other and to the columns. The level of the roof is adjusted by the connection between the hanger and the roof.

The deflection of the rods under the appropriate load was calculated and, by comparing the position of the rod to a string line, the adjustment can be controlled sufficiently accurately. The columns are erected with the rods attached.

The adjustment is provided by welding left and right hand threaded sections to the rods and connecting them with an internally threaded circular bar which is notched for a spanner at one end. The tie-downs are also adjustable.

All of the structural connections necessary for extension can be made before removal of the cladding. The same column and beam details are used throughout except for the addition of a plate for the push-pull brace and an extra plat on the top of the perimeter columns, and so the addition or substitution of elements necessary for extension is simple. Loads from the additional structure can be transferred to the existing column before removal of the tie-down, thus ensuring equilibrium.

4.0 Wall structure

The external walls are set inside the perimeter column line. The structure to the roof passes through the wall and is exposed on the perimeter. The walls are made to two skins of profiled steel sheet with insulation fixed on vertical lightweight steel trusses at 2 m centres. The top of the wall is held in an inverted U detail which permits the roof to deflect vertically and allows relative horizontal thermal movement to take place.

The inverted U forms the base to a window which runs as a thin band around the perimeter at high level. As well as separating the walls from the roof visually and lighting the interior, this provides a zone for the structure to penetrate the wall in a way which makes explicit the relationship between internal and external structure. This window is particularly effective at night. Wind forces from the walls are taken by horizontal trusses around the perimeter. These trusses are bolted up under the main roof steelwork in 18 m sections which may be removed for re-use when the building is extended. The trusses transfer the wind forces to the main column lines where they are taken out on external raking tubes. These push-pulls are provided on two adjacent sides only, to allow for free thermal expansion.

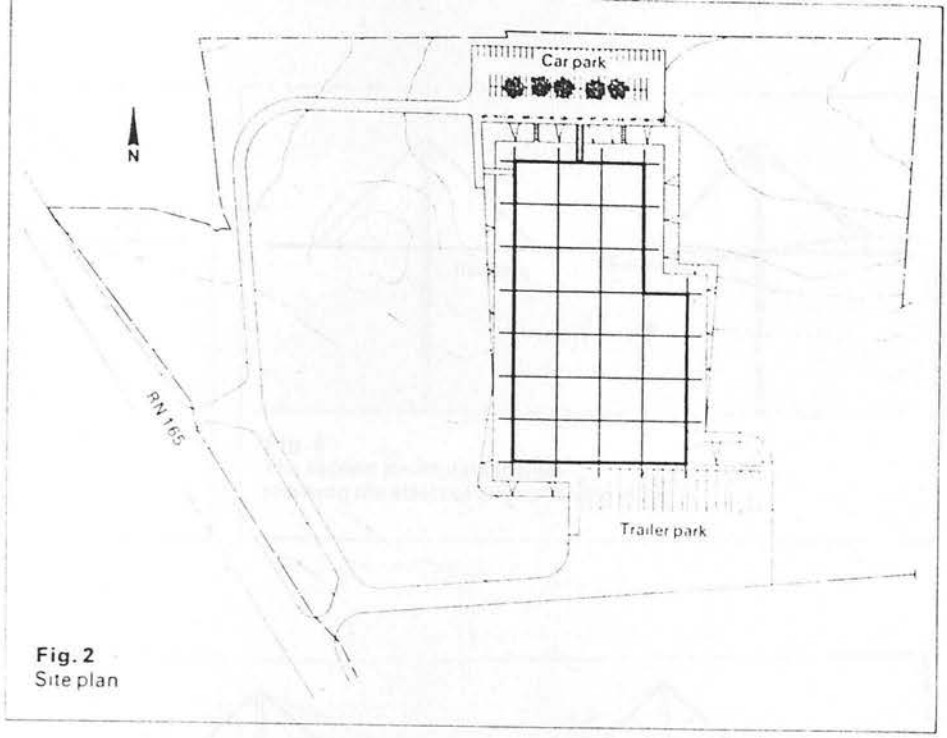


Fig. 2
Site plan

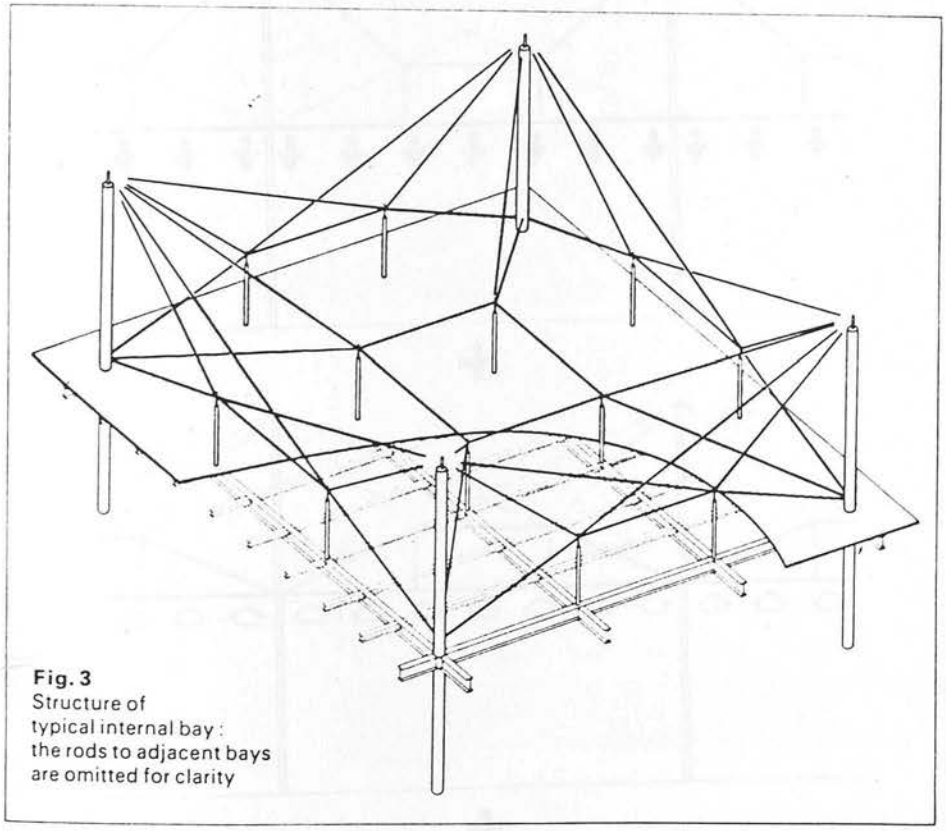


Fig. 3
Structure of typical internal bay: the rods to adjacent bays are omitted for clarity

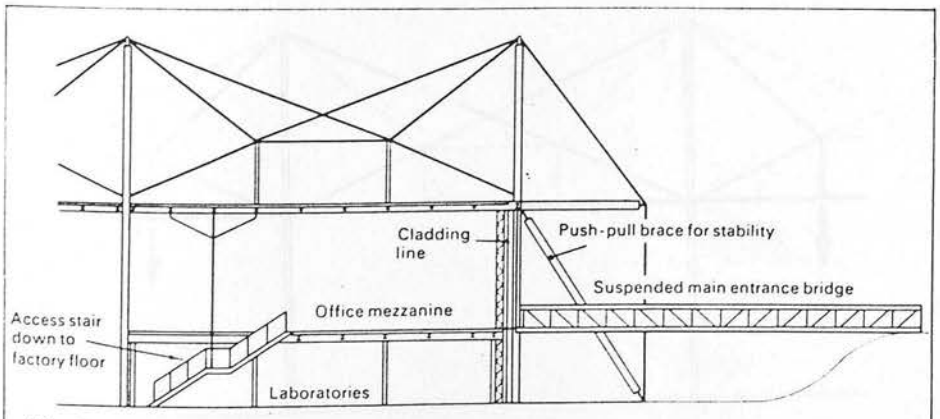


Fig. 4
Section through the central bay of north elevation showing the main structural features

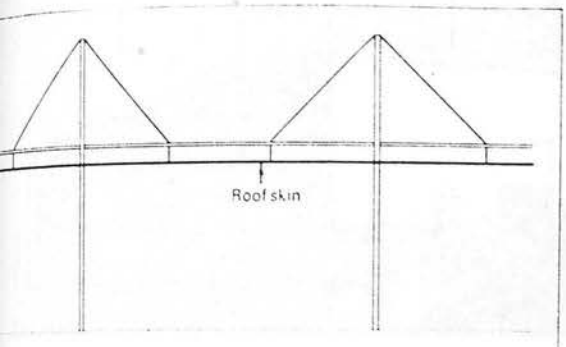


Fig. 5
The first
structural scheme

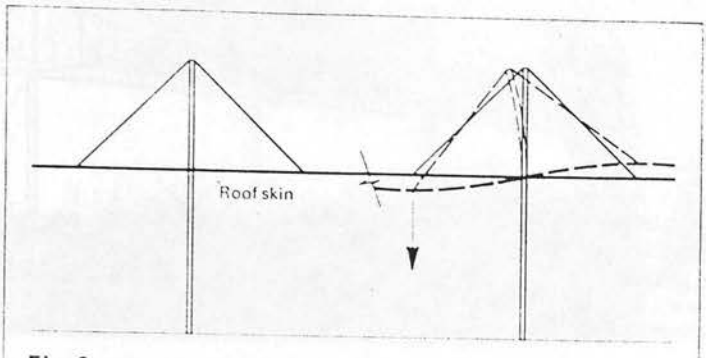


Fig. 6
The second structural scheme
showing the effect of unbalanced loading

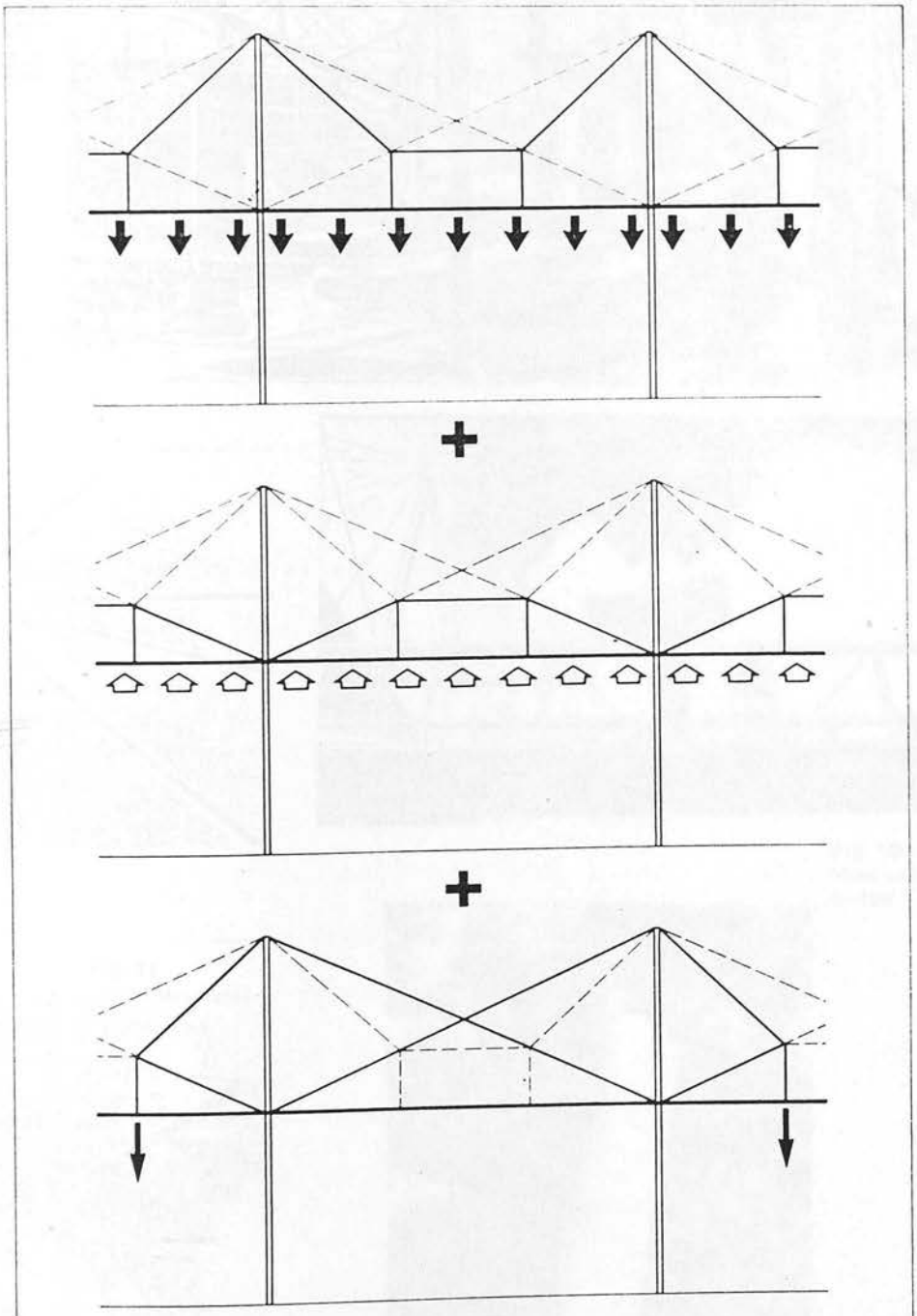


Fig. 7
The three primary load carrying systems
within the overall roof structure

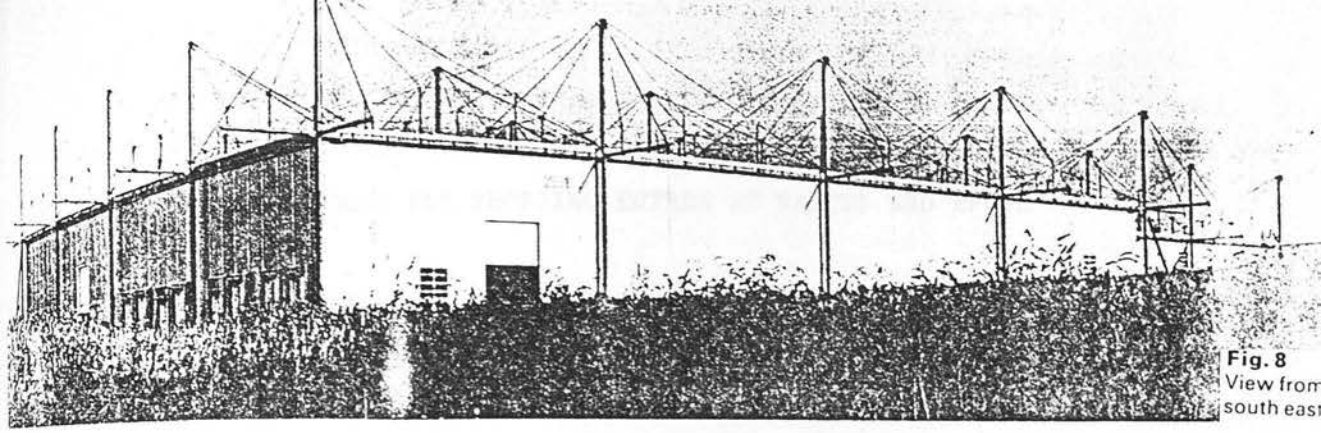


Fig. 8
View from south east

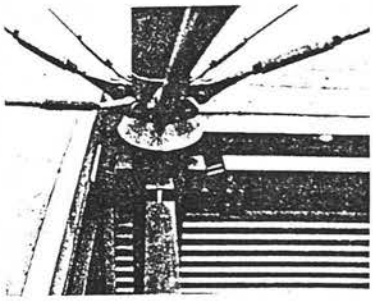


Fig. 14
Column detail at roof level

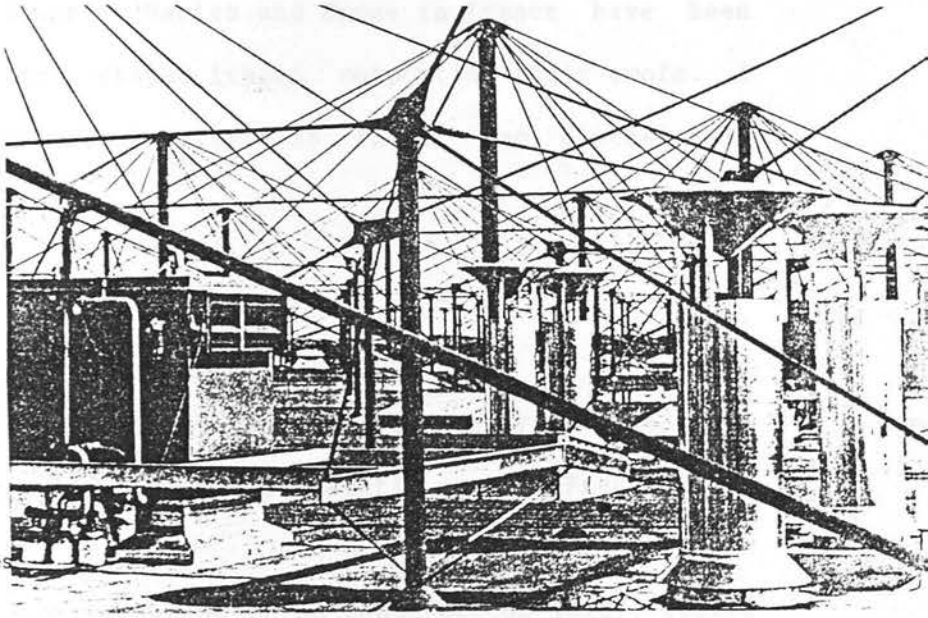
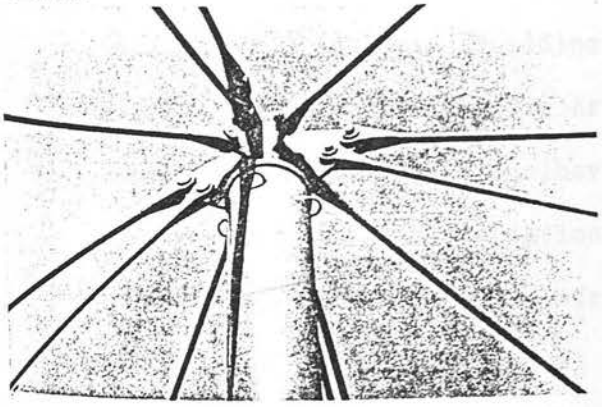


Fig. 9
Roof scape with flues and suspended air handling units

Fig. 12
Column head details. The rings are for use during maintenance.



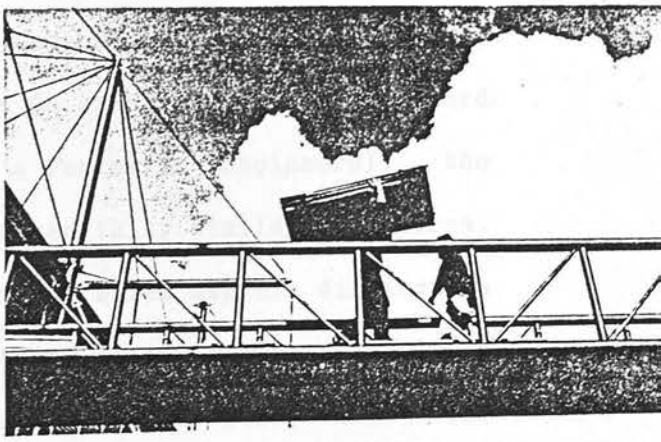
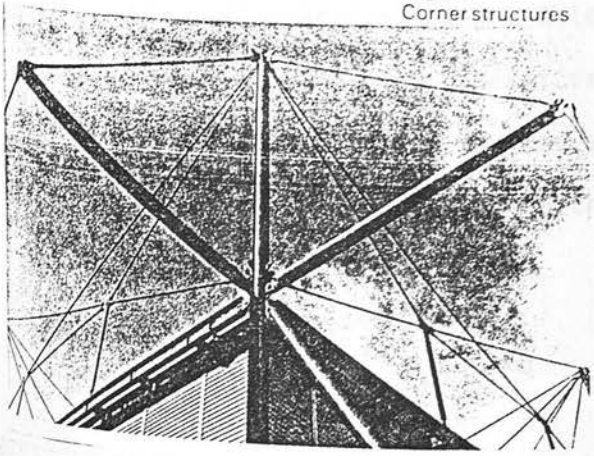


Fig. 10
Main entrance bridge

Fig. 11
Corner structures



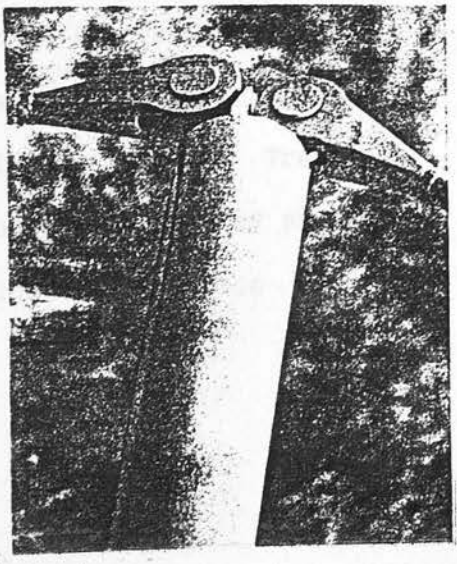


Fig. 13
Boom end de

CASE STUDY No. 17

CABLE STAYED ROOFS FOR SHOPPING ENTRES AT NANTES AND EPONE

Extracts from a Paper by
P.R. Rice, J.A. Thornton and E.A.R. Lenczner
Ove Arup & Partners, London

Two new shopping buildings at Nantes and Epone in France have been designed with external cable stayed frames supporting their roofs. A distinctive feature of the structures is their 'two dimensional' transverse from systems.

1.0 Introduction

The Usine Centre, St Herblain is a shopping centre housed in a single building at Nantes, near the Atlantic cost of France.

The client, Groupe de Recherche et de Construction (GRC), wanted the buildings to have distinctive external structures. Similar to the 1980 Fleetguard building at Quimper designed by Richard Rogers, (Architect) and Ove Arup & Partners (Engineers), the building at Nantes and Epone have essentially similar structures, as was intended, but variations in detailed aspects distinguish each from the other. The structural design can be seen as a progression from that used at Quimper.(case study 16). The principle change is that the new buildings have a series of independent parallel, external support frames instead of an intersecting grid pattern. This allows the new roof structure to be seen as clearly identifiable planar frames. Transverse horizontal stability at Nantes and Epone is provided by pin-based frame action as opposed to the inclined prop action used at Quimper (Fig. 3).

2.0 Description

The overall plan size of the building at nantes is 129.6 m long by 86.4 m wide, whilst that at Epone is 403.2 m by 86.4 m. Whereas the plan shape at Nantes is rectangular the ends of the Eptone building are cut back in a stepped pattern (Fig. 4). The enclosed building volume is divided into two levels by a precast concrete first-floor structure. The external steel structure supports the roof and provides horizontal stability to the upper part of the external walls.

The primary roof structure consists of parallel transverse frames at 14.4 m centres. the standard three-span frame has four 28.8 m tall tubular masts at 28.8 m centres. At Nantes the mast diameter is 355.6 mm whilst at Epone it is 457.0 mm. the masts are linked at mid-height by a line of transverse span-booms (tubes, diameter 355.6 mm). Outrigger booms (tubes, diameter 275.0mm) on the same line extend beyond the outside masts. A planar system of steel tie-rods (diameter 45 mm) and strut/ties (tubes, diameter 88.9 mm) is attached to the masts and booms to complete the transverse frome (Fig. 5). The stepped ends of Epone incorporate one and two span versions of the transverse frame.

Steelwork connections at the node points within the transverse frames are made using pin-axle assemblies with profiled transfer plates (Fig. 6). Tie-rods have specially made fork-end terminals to fit the transfer plates. Adjacent frames are connected together by longitudinal booms which attach to the masts at their mid-height and form a rectangular grid pattern with the

transverse booms.

The masts at Nantes are provided with a secondary rod bracing system to give additional resistance to out-of-plane forces. Those at Epone are of larger diameter but have no secondary bracing. The masts at Epone also have helical strakes at the top. those portions of masts within the building enclosure are filled with conrete to make the tubular section fire resistant.

Longitudinal roof beams, with 'I'-sections (420 mm deep, 180 mm wide) span 14.4 m between transverse frames. They are suspended by tubular hangers about 1.5 m below the transverse booms at 4.8 m centres. Insulated profiled metal decking roof is fixed on top of these roof beams and the roof construction completed with insulation and waterproofing. A roof pitch of 5 degrees for drainage is achieved by varying the length of the hangers from the booms.

Overall transverse stability is provided by frame action of the main transverse system. Longitudinal stability is provided by braced transverse bays. Cross-bracing occurs between pairs of transverse frames in the plane of the outside columns. This cross-bracing is continued over the roof by linking the transverse booms to form a horizontal transverse girder (Fig. 7). the bracing is thus in the form of an inverted 'U' which straddles the building. Two bays are braced at Nantes and five at Epone. The cross-bracing in the girder is arranged in the form of intersecting shallow diamond frames to span the diagonal distance between frames.

The external walls are deep-profiled steel sheeting which span from ground to roof with intermediate support from the first floor. These walls are set 2.4 m inside the outer column line. At the entrances, the profiled sheeting is replaced by full height screens supported on vertical steel trusses. Steel footbridges span across these areas at first floor level. These bridges are supported on longitudinal frames consisting of continuous 'I' beams stiffened by tie-rods below. At Nantes entrance to the building is gained via another external bridge which connects to the internal footbridge through the glazed screen.

Fire escape routes from the first floor levels of the buildings are via external steel staircases. These fit within the space between the outside column line and the facade.

3.0 General Structural Behaviour

3.1 Vertical loads

The transverse frames provide all the vertical support to the roof of the building. They are designed to resist three types of vertical loads:

i) uniform gravity loads
ii) patch and concentrated gravity loads
iii) wind uplift.

The concept of the tie-rod arrangement over the 28.8 m spans was to provide clearly identifiable systems to resist each of these types of loads. Overall vertical support for distributed loads

is provided by the long ties attached to the tops of the inside and outside masts. Patch and point loads are redistributed by the triangulated tie system above the transverse booms in conjunction with bending of the booms. The tie-rod system incorporates an effecitve tension chord which works in conjunction with the compression boom to provide resistance against uplift. The vertical elements within the system which tend to go into compression in the wind uplift condition are made as tubular struts.

The vertical loads, are applied to the transverse frame at boom level. The tie system is attached to the boom and to the mast heads. Because the resultants of the tie-forces at the mast heads are contained within the plane of the tie system, their lines of action are always contained within the plane of the system above boom level. Thus displacements of the mast head do not cause the line of resultant compression force on the mast to deviate from the mast axis at boom level. The buckling length of the mast is therefore equal to the mast length between boom-level and mast head. the buckling length would not be reduced by using out-of-plane ties attached to the mast-head.

The rod system is provided with adjustment in some ties to compensate for lack of fit and take out a slack. The system was not designed to be prestressed othe than by dead-weight.

3.2 Transverse forces

3.2.1 Overall

The transverse frames also provide transverse

horizontal stability.

As shown in Fig 8, the tie-down from the outrigger boom returns to the foot of the external mast rather than passing straight down to ground thus competing an effective portal frame capable of resisting horizontal loads parallel to the frame. Another benefits is that the uplift from the tie-down reacts directly against the mast reaction and hence the size of the total mast base reaction is reduced and the need for a separate tension anchorage is avoided. This is particularly important at Epone since the high water table means that extra anchorage mass is required to compensate for buoyance effects. The details of the base of the external mast, with tie connections, is shown in Fig 9.

3.2.2 Wind on side facades

The two outermost longitudinal roof beams along each side are braced together to form eave girders which transfer the wind load from the walls to the transverse frame line. A series of tubular struts, between roof beams, tranfer these loads to the centre of the building where they pass to boom level via stiffened hangers. This arrangement accommodates transverse thermal expansion from the centre without the need for expansion joints. If the stiffened hangers were placed directly above the eave girders differential thermal expansion between the external structure and the

suspended roof would have generated unacceptably large moments in the stiffened hangers.

3.2.3 Transverse frame stiffness

The overall in-plane stiffness of the transverse frame depends primarily on the axial stiffness of its triangulated constituent members. The bending stiffness of the mast and boom members also contribute to the overall stiffness but to a lesser extent. The vertical span deflections and horizontal sway displacements are therefore principally controlled by the cross-sectional areas of the various elements.

The sizing of the mast and boom elements is governed by their buckling and bending capacities. The tie-members however, are sized to provide sufficient stiffness to the frame. The comparative importance of an individual member's contribution to the overll stiffness depends on its position within the system. The longer and more highly loaded members contribute most to th total displacements. Thus the ties connecting the base and head of the outside masts to the end of the outrigger are important because they are comparatively long and highly loaded, and so were given a larger corss-section than the other ties (diameter 65 mm instead of 45 mm). It was decided not to use more than two sizes of tie for the planar system in order to minimize the number of different types of fork-ends and adjusters required.

3.2.4 Out-of-plane forces on transverse frames

Early analysis showed that the masts were susceptible to wind effects out of the plane of the transverse tie system. One the one hand, with wind blowing parallel to the planar system, the mast could start vibrating due to oscillating eddy-shedding around the mast. On the other hand, with wind gusting perpendicular to the planar system, significant moments could develop in the mast amplified by dynamic and geometric second-order effects. Mast-head deflections could also become unacceptable large.

Both effects were linked to the natural response frequently of the mast which had to be computed. The dynamic magnification factor used for design wind forces on the mast depended on the mast frequency. A non-linear analysis had to be used to analyse the moment amplification due to geometric second-order effects (see later section).

Different methods for coping with the above phenomena were adopted at Nantes and Epone. At Nantes the natural frequency of the mast (diameter 355.6 mm) was increased to avoid dynamic problems by using light ties out-of-plane to enhance the mast stiffness (Fig. 10). This avoided the need for helical strakes. At Epone it was decided to allow a larger diameter mast (457.0 mm) sufficient to resist the amplified moments and prevent excessive deflections; however, helical strakes were

necessary to deal with the vibrations due to eddy-shedding.(Fig. 11).

3.3 Longitudinal forces

3.3.1 Overall

Overall longitudinal stability is provided by the cross-braced inverted 'U' frames described earlier.

3.3.2 Wind on end-facades

Longitudinal wind loads on the end-facades of the building are collected by an eave beam and then taken up to boom level by the roof beam hangers. The loads are transferred to a transverse braced bay via column line longitudinal booms and extra intermediate mid-span booms which exist only between the end facades and the over-head girders.

There is an expansion joint in the external longitudinal booms between braced bays. At Nantes expansion and contraction in the roof beams betwen the two braced bays is absorbed by movement in the bolt holes and flexure in the beam hangers. At Epone, where the length between extreme braced bays is over 250 m, explicit joints are provided between baraced bays.

3.3.3 Lateral stability to booms

Lateral stability to the transverse booms is provided

necessary to deal with the vibrations due to eddy-shedding.(Fig. 11).

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3.3.3 Lateral stability to booms

Lateral stability to the transverse booms is provided

by the bending stiffness of the hangers and beams which support the roof. Relative lateral displacements of the successive hangers along the boom is prevented by the diaphragm action of the profiled metal decking fixed onto the roof-beams.

3.3.4 Roof decking

The profiled metal decking spans continuous over the roof beams and, along each side-facade cantilevers to form a 1.8 m canopy overhang. The decking is used to provide stability to the flange of the roof beams as well as the main transverse booms.

The in-plane shear stiffness of the metal roof deck is sufficient for the decking to provide significant resistance by diaphragm action. However, to simplify the contractual relationships between the various contract packages and satisfy the demands of the French checking authority it was necessary to provide explicit structural systems for all loads within the steel framework. An exception was made for the stabilization of the booms and roof beams which was provided by the decking.

4 Special analysis

4.1 There were certain aspects of the behaviour of the transverse frame system which required special investigation using non-linear computer analysis:

In-plane-effects:

1) The effect of the tie-members within the system going stack and effectively losing their axial stiffness under compressive strains had to be allowed for in the inplane analysis of the transverse frame.

2) The effects of self weight of the tie-members causing sag had to be considered.

Out-of-plane effects:

3) Effects due to non-planar geometric imperfections, were studied.

4) Out-of-plane effects due to loading components perpendicular to the plane of the frame had to be analysed with particular regard to the second order effects associated with large displacements.

Conventional computer analysis programmes are not properly able to take into account all these phenomena and so a non-linear type analysis programme was required.

4.2 Construction method considerations

The construction sequence for the catenary systems on site would mean that by the time the final tie-members were installed just above the boom position the steel-work already erected would have deflected under tis self-weight. It was proposed that those ties with adjustment just above the boom would be made taut after the roof construction was completed. The adjustment facility in

these ties was necessary both to allow for dead weight deflections as well as fabrication tolerances on element lengths. The amount of adjustment required to take up slack caused by dead weight deflections was well within the amount required for fabrication tolerances.

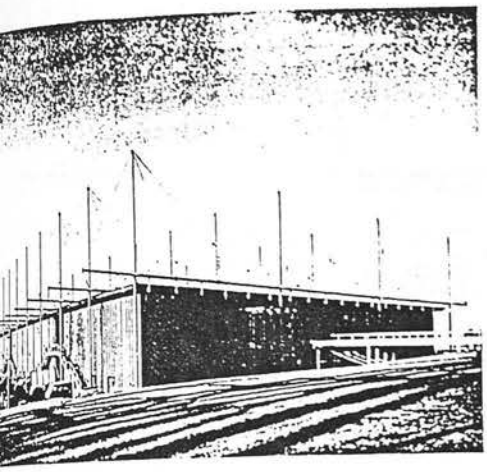
4.3 Effect of self-weight sag

The self-weight of a tie spanning a horizontal distance causes it to sag. Tie tension is a function of its sag. If the tie bars are represented by single elements, the computer model does not take this sag into account and the tie elements have the stiffness of a straight bar. However to model each tie as a chain of elements with self weight applied at nodes along their length means each analysis becomes significantly larger and more expensive.

A comparative study was made between the results of an analysis made each way for an individual load case. The results showed that there was no significant differences in the overall distribution of forces within the frames or its stiffness. It was found that for their operating range of tensions the sags in the ties were not large enough to make the effective tie stiffness, as it affects the whole system, significantly different from a straight bar. The most important ties, those which carry the heaviest loads, were those least affected.

When the distance between the ends of a tie is less than the straight length of that tie, the self weight still produces a tension. Therefore the tie will retain residual stiffness due to

this 'built-in' pre-stress effect.



1. View of Centre Commercial at Nantes.

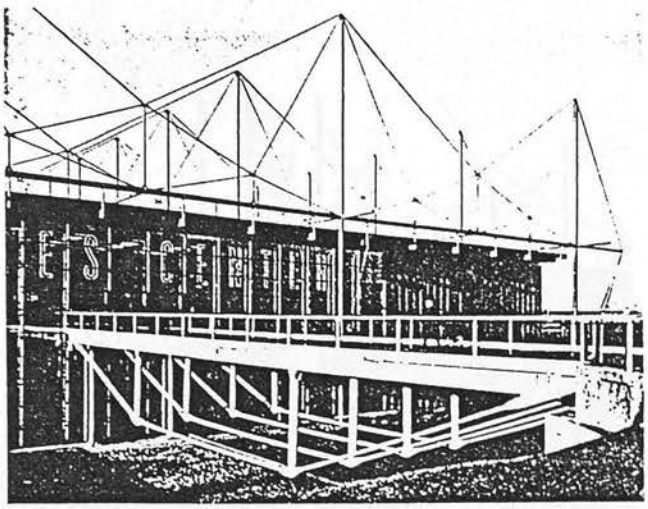


Fig. 2. Entrance Facade of the Centre Commercial, Nantes.

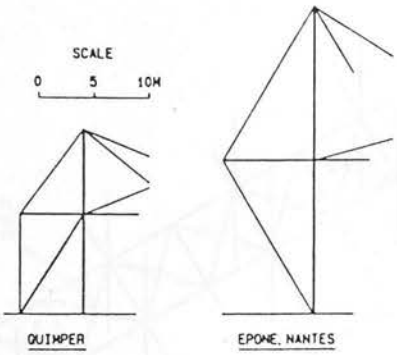


Fig. 3. Comparison between tie-down arrangements at outer masts. Quimper: tie passes vertically to ground and inclined prop provides horizontal stability. Epone, Nantes: tie returns to base of outside mast to complete portal frame system resisting horizontal loads.

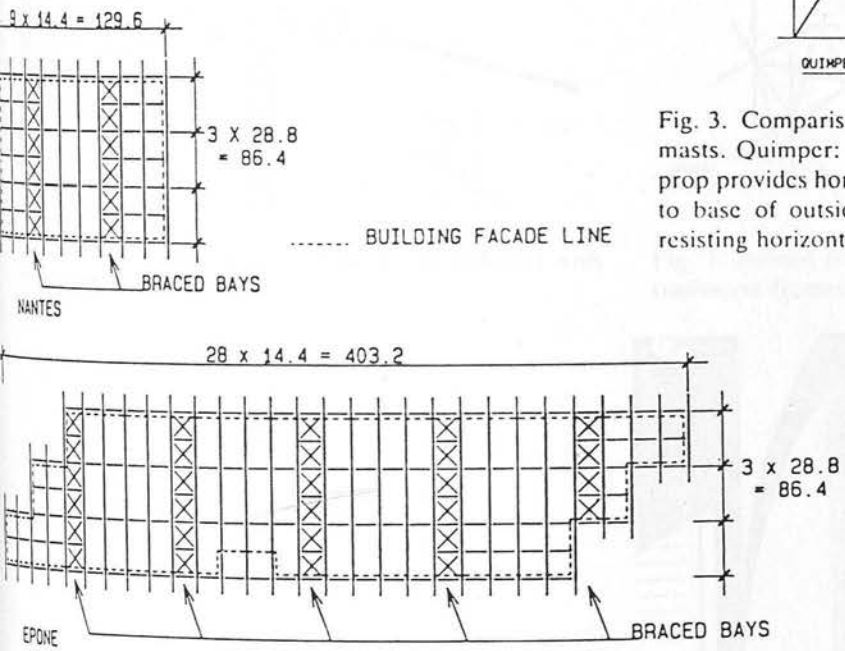


Fig. 4. Plan arrangements of buildings.

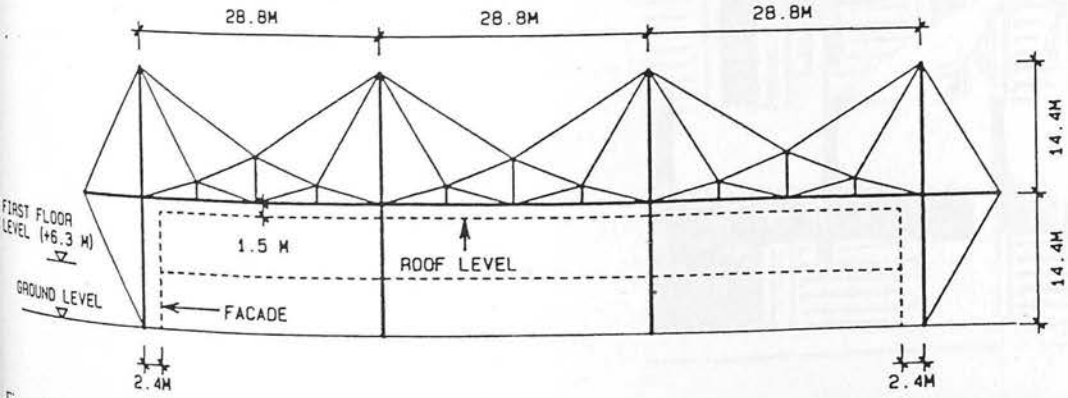
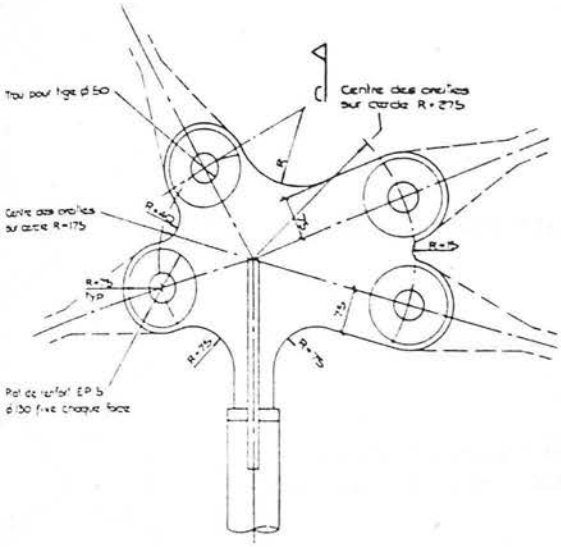


Fig. 5. General arrangement of 3-span transverse frame.



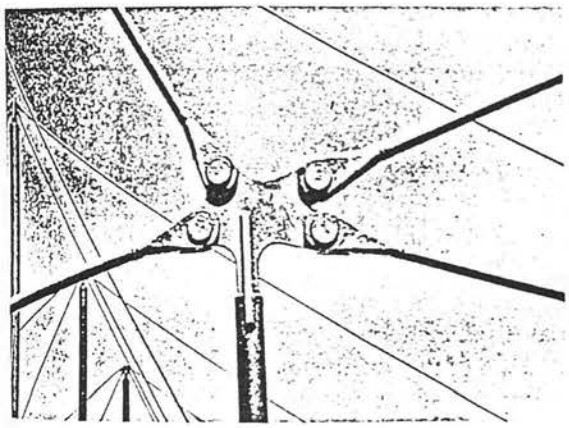


Fig. 6. Details of tie node transfer plate and fork-end with photograph of assembly on site.

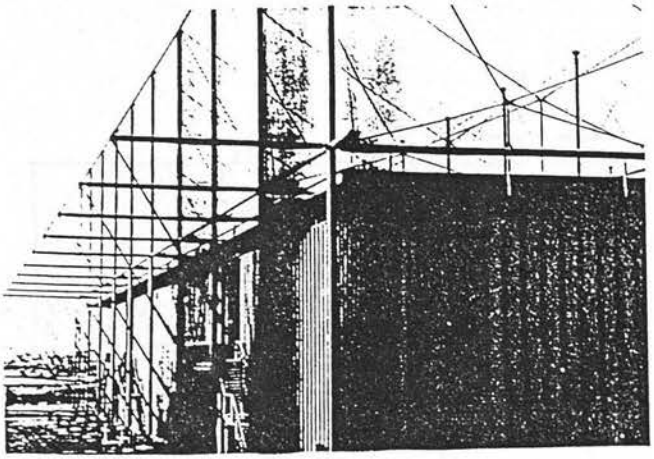


Fig. 8. View along side of building showing 'outrigger' booms and braced bays.

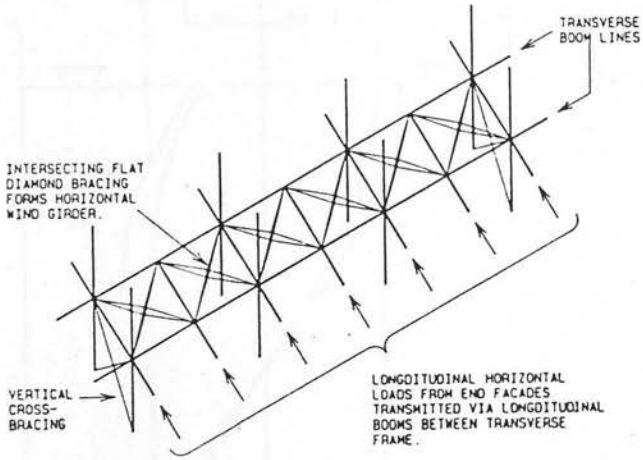


Fig. 7. Braced traverse bay: schematic diagram (ties within transverse frames omitted for clarity).

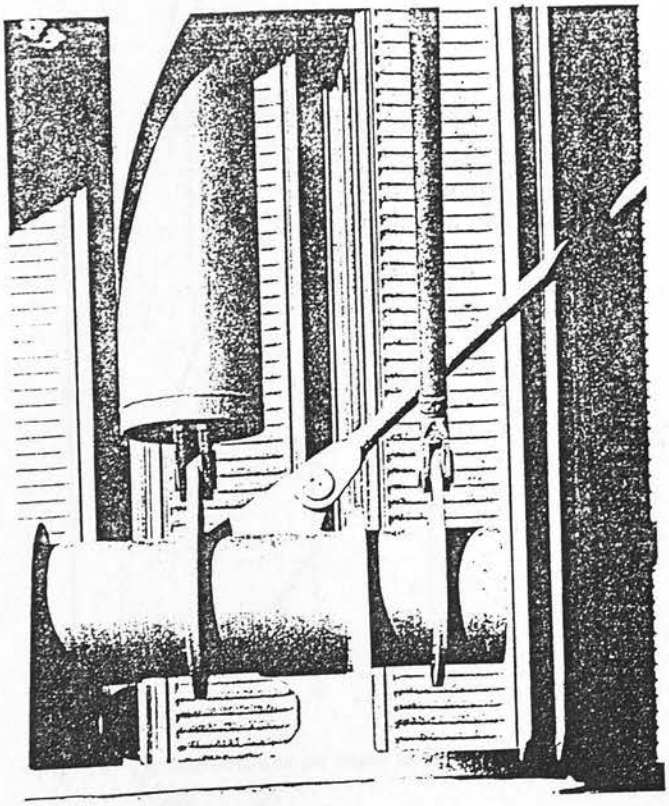
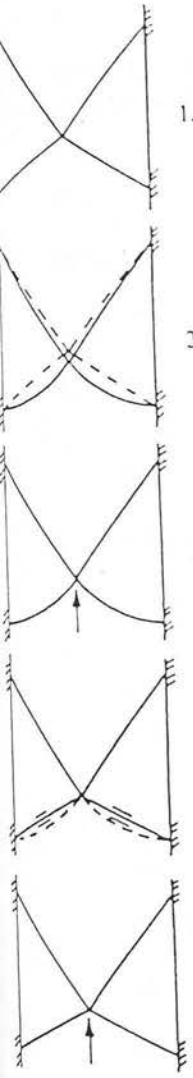


Fig. 9. Detail at external mast base with tie connections.



1. Tie system erected without gravity effects.

2. Tie system displaces under self-weight gravity loads. Lower ties go slack.

3. Lower ties do not immediately act to resist upward load, slack must be taken out first.

4. Lower ties adjusted to take out slack caused by self-weight gravity effects.

5. Lower ties now act immediately to resist upward load and increase system stiffness.

Fig. 12. Self-weight effect on simplified tie system.

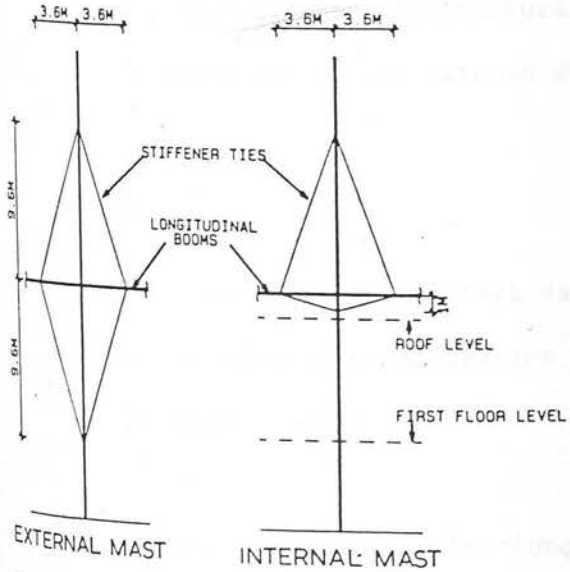


Fig. 10. Out-of-plane mast stiffener systems used at Nantes.

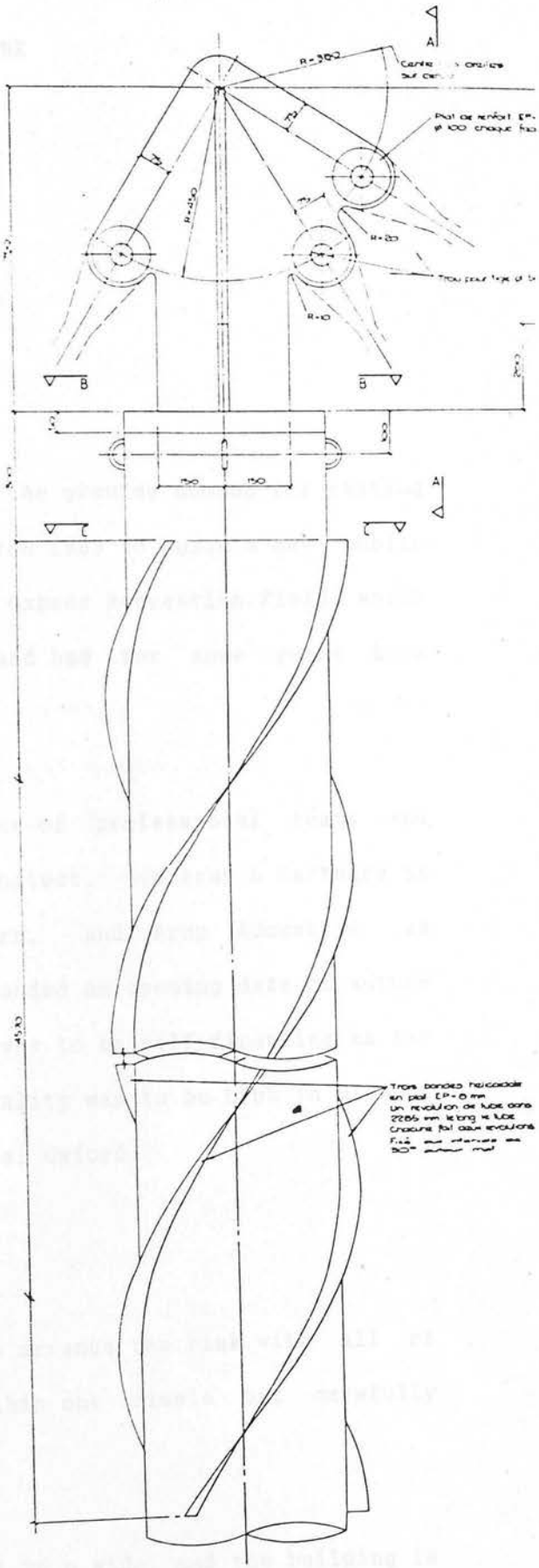


Fig. 11. Helical strakes on mast at Epone.

CASE STUDY 18

OXFORD ICE RINK

Extracts from a paper by
Philip Dilley, OAP

Engineers: Ove Arup & Partner

Architects: Nicholas Grimshaw & Partners

1.0 Introduction

The Oxford City Council recognised the growing demand for skating facilities and they decided in March 1983 to build a new public ice rink. The site was to be the Oxpens Recreation Field, which is close to the city centre, and had for some years been earmarked for a sports complex.

The Council interviewed a number of professional teams and appointed Nicholas Grimshaw as architect, Ove Arup & Partners as structural and services engineers, and Arup Acoustics as acoustics advisors. The brief demanded an opening date of autumn 1984, and the required building was to be self-financing as far as possible. The architectural quality was to be high in view of the proximity of the site to central Oxford.

2.0 Concept

The architectural approach was to arrange the rink with all of its ancillary accommodation within one simple but carefully designed envelope.

The ice pad itself is 56m long and 26 m wide, and the building is

72 m x 38 m overall. Most of the public facilities are housed at one end of the space, both at ground level and on a first floor mezzanine.

It was considered vital to the success of the rink in terms of attendance levels and income that not only should the design be technically satisfactory with the ability to provide ice conditions and comfort levels appropriate to each activity, but that the rink should also project an image or identity of its own. It should be fun to visit and be immediately recognizable - perhaps even controversial should not be by passed without notice. (Fig 1, 8, 9)

With this in mind the ice pad was positioned as close to Oxpens Road as sensibly possibly, and a glazed end wall is provided to enable the public to see into the ice hall. At night, the disco lights and lively activity provide a spectacular advertisement to the passer-by. The transparency of the end wall was an expression appropriate to the arrangement of the structure, as will be described later.

The main entrance is at first floor level and is approached along an external ramp (Fig 6). With most of the ancillary facilities at mezzanine level, non-skaters have immediate access to the bar, creche, skate shop and spectator seating. Only skaters need proceed downstairs to the skate hire shop and changing areas.

Spectator 'bleacher' seating is arranged along each of the long sides of the ice pad so that when not in use it can be retracted

to enable skaters to pass around the outside of the ice barrier, and use the fixed benches which are otherwise concealed. Access to the spectator seating is from a first floor gallery which forms an extension to the mezzanine at each side of the rink.

Externally, it was felt architecturally essential to avoid the warehouse-like appearance which this volume could easily generate. Several forms of external structure were considered to provide an economic but visually interesting appearance.

3.0 Substructure

A site investigation established that the site had been used in Victorian times as a refuse tip, and had not previously been developed. Since then, it had been grassed and used for many years as a recreation space. The 2 m depth of Victorian fill was generally inorganic but contained local pockets of large numbers of discarded bottles. Beneath this fill was a further 1 m of soft alluvial clay, 4 m or so of water-bearing gravel and then Oxford clay.

Although the gravel would provide a good bearing for shallow foundations, excavations through the soft clay and into the ground water would be expensive and, just as important, slow. The options were then to found in the fill at very low bearing pressures, or to pile through the gravel into the Oxford Clay. The adopted design used a combination of both of these. This will be described in the section on the superstructure.

4.0 Superstructure

A steel frame, described in more detail later, provides roof beams at 4.8 m centres and edge columns 1 m within the perimeter of the building. The mezzanine is an independent steel structure with its concrete floor poured on Holorib profiled steel deck.

The cladding and roofing is designed to give high values of thermal and acoustic insulation, as well as being robust and fast to erect.

A structure to cover a volume such as this could be easily and economically provided by a simple portal or braced frame. However this would not satisfy the architectural and planning needs already discussed, and various forms of external structure were considered. The aim was to design a frame which would contribute to the building's deliberate nautical image, yet not at any substantial cost penalty. Whilst fairly generous fabrication time was available during the construction of the ice pad ground works, it was essential that the steel components were easily handled and delivered, and fast to erect.

The chosen solution uses a central longitudinal spine beam made from a pair of rectangular hollow sections, and spanning up to 15 m between overhead supports. This then halves the span of the transverse roof beams which are continuous over the spine but pinned at their ends. (Fig 4, 7)

The suspension arrangement uses the spine beam as a principal compression member with its twin RHS passing either side of the

main masts, and has sets of four high strength stainless steel bars as the tension elements. The tie bars in the planes perpendicular to the spine beam simply restrain the mast which have pinned bearings at each node. This geometry results in a large vertical load at each mast position (380 tonnes) and a smaller uplift at the anchor points. These loads are carried on straight shafted piles bore into the Oxford Clay, whereas the perimeter columns, carrying only 20% of the roof load are founded on shallow pad footings in the fill, all resulting in a particularly economic foundation solution.

5.0 Wind effects

A detailed study of wind effects on this building was undertaken to ensure that wind-induced vibrations would be controlled, and that wind uplift pressures would not affect the structural stability.

Wind uplift forces derived from CP3 do not exceed the dead weight of the roof and reversal of stresses in the tension bars apparently would not therefore occur.

So far as wind-induced vibration of the tie was concerned, calculations for the highly stressed bars suggested resulting amplitudes of vibration which may give rise to fluctuating bending stresses and which, in the long-term could lead to fatigue. To limit the growth of these vibrations, dampers have been attached to all but the shortest of the tie bars. These devices are 'Stockbridge' dampers usually seen protecting transmission lines from damaging vibrations.

6.0 Construction

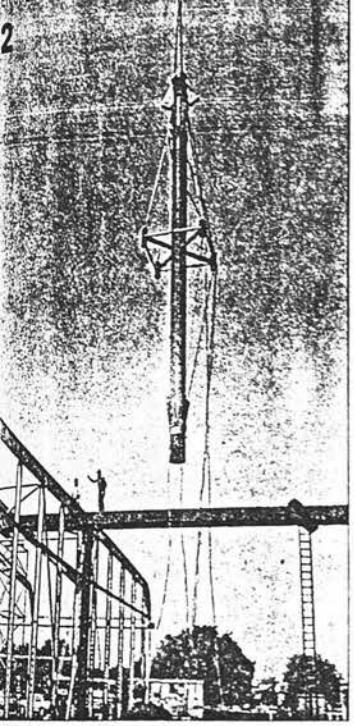
It is worthy of note that despite the dramatic appearance of this steelwork, over two-thirds of the steel weight is entirely conventional with simple fabrication, and the only complex and expensive joints arise at the tie bar connections, and mast bearings.

Ease of erection of the steelwork follows a similar pattern, with the complicated procedure concentrated in small areas of the work. The spine beam was delivered to site in five road-transportable sections and supported on the lower mast lengths, and on temporary support towers. The junctions were then welded on site to produce the continuous member needed to carry the large compression forces.

The remainder of the frame and envelope could then proceed traditionally, with the masts and tie bars being applied outside the critical path of the programme.

Each section of the masts needed three cranes for its erection; one to suspend the mast whilst two others manoeuvred the tie bars into position in their purpose-designed temporary cradles. (Fig 2, 3, 9) Final adjustment was made by jacking the bars using 'Pilgrim' nuts , to achieve the prescribed geometry. One inevitable consequence of an external structure is that components must penetrate the roof. This occurs only four times here, and care has been taken to prevent cold bridges at these points.

2



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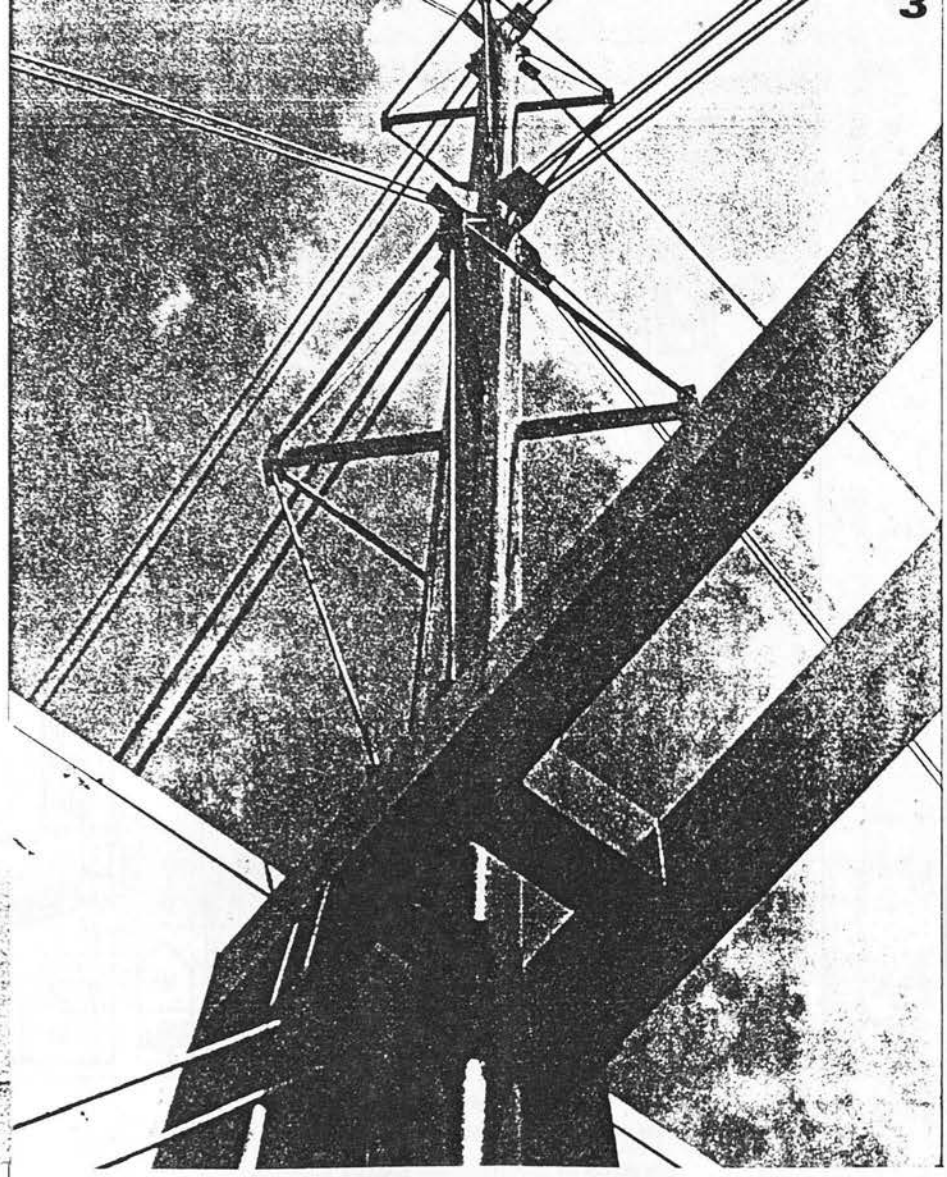
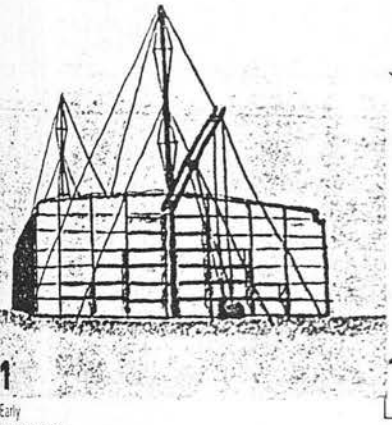
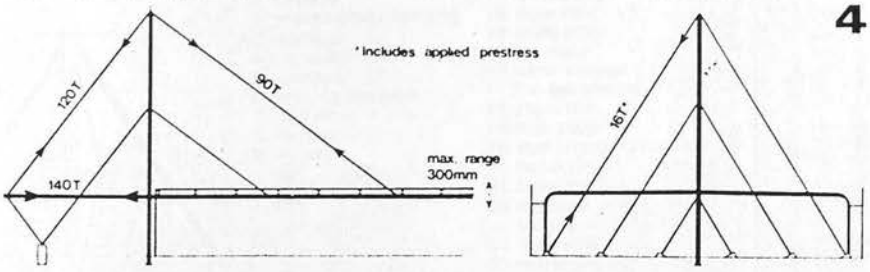




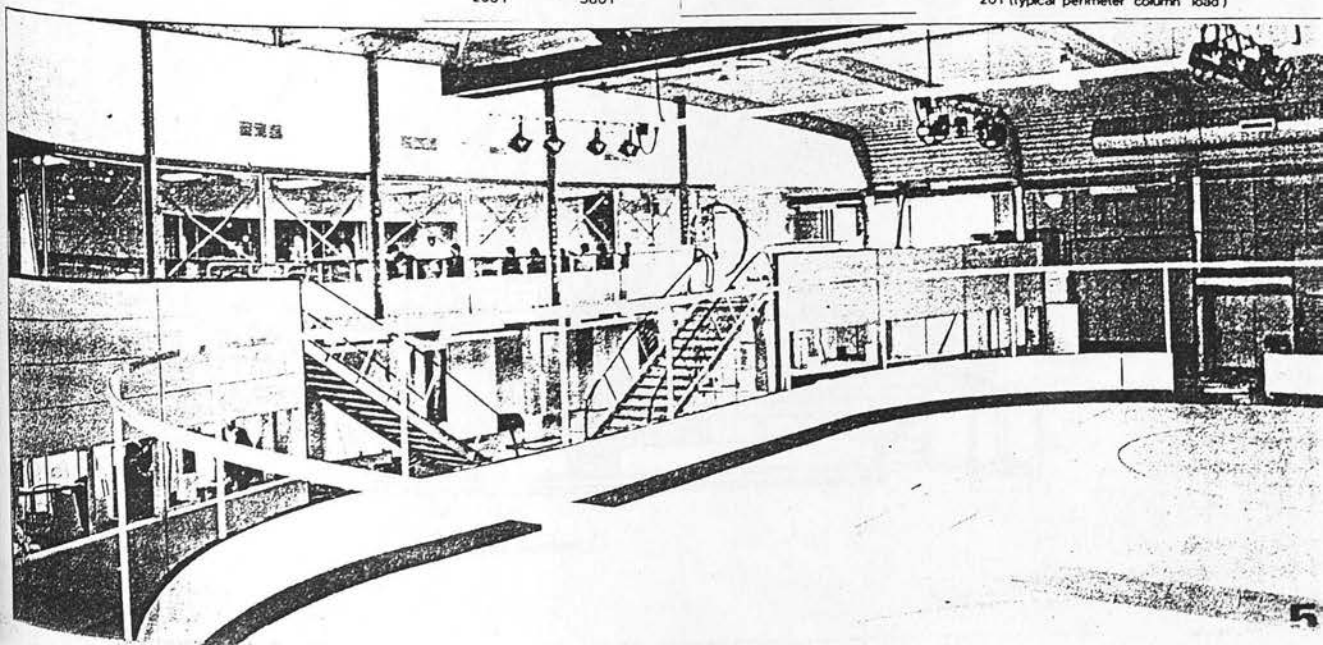
1

Early architectural image

4



Structural design forces



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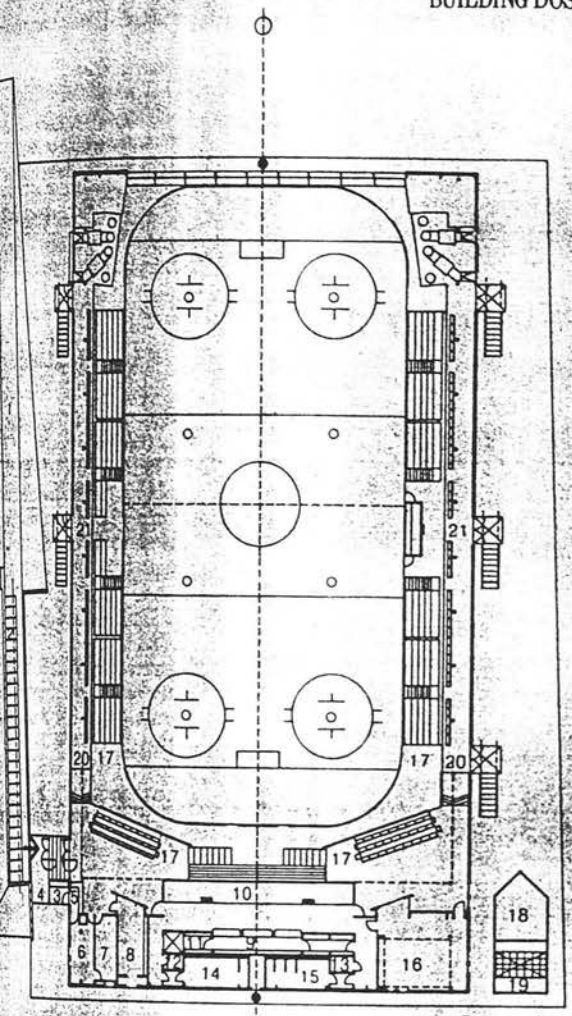
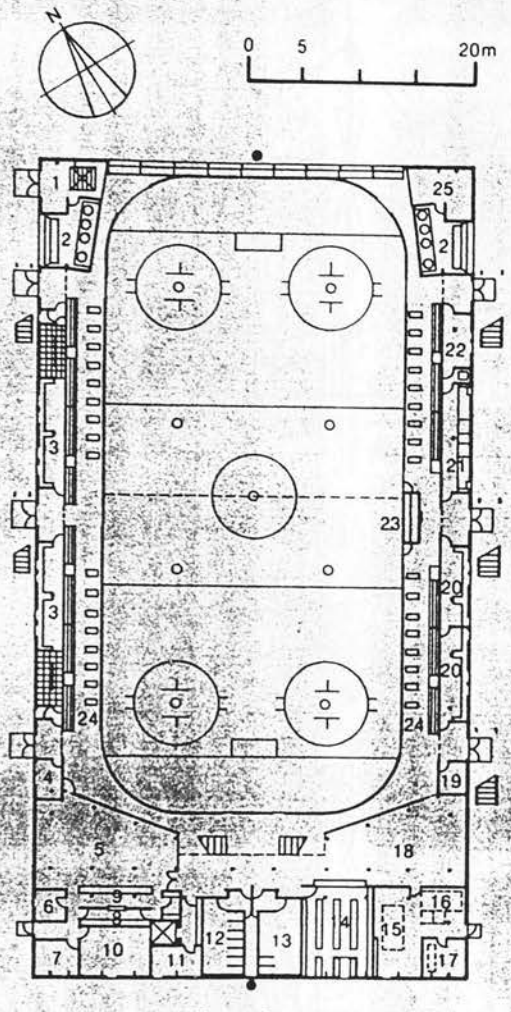


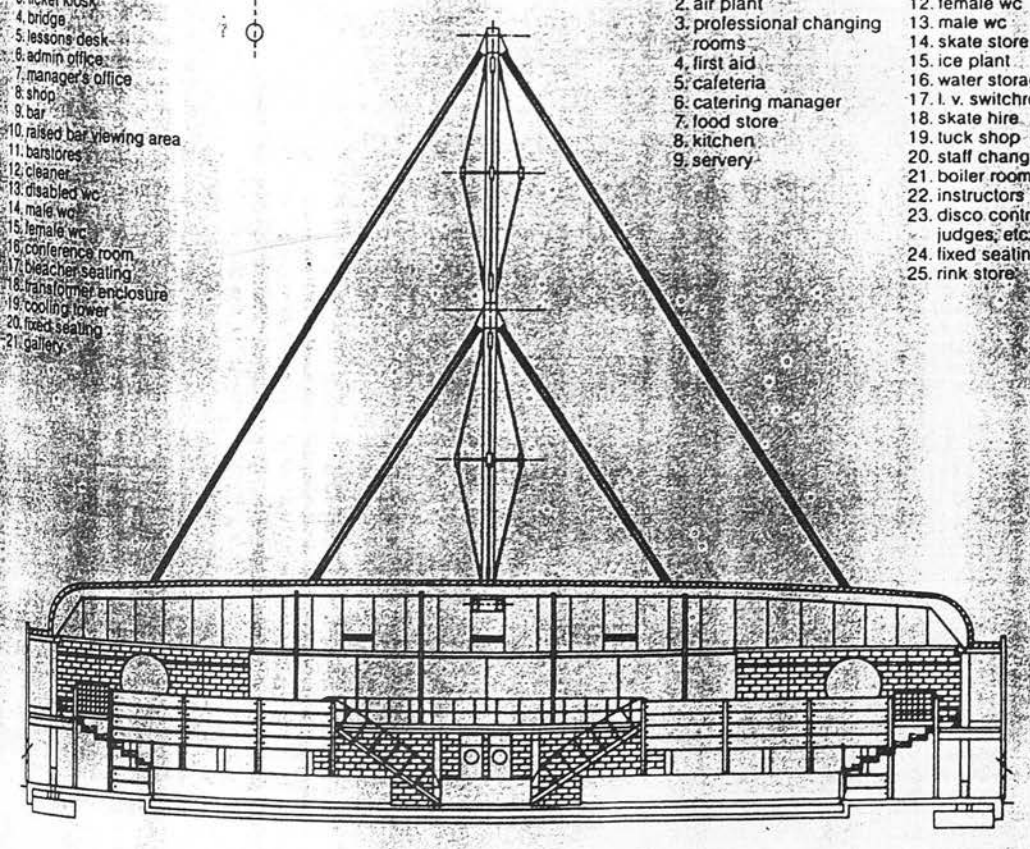
First Floor



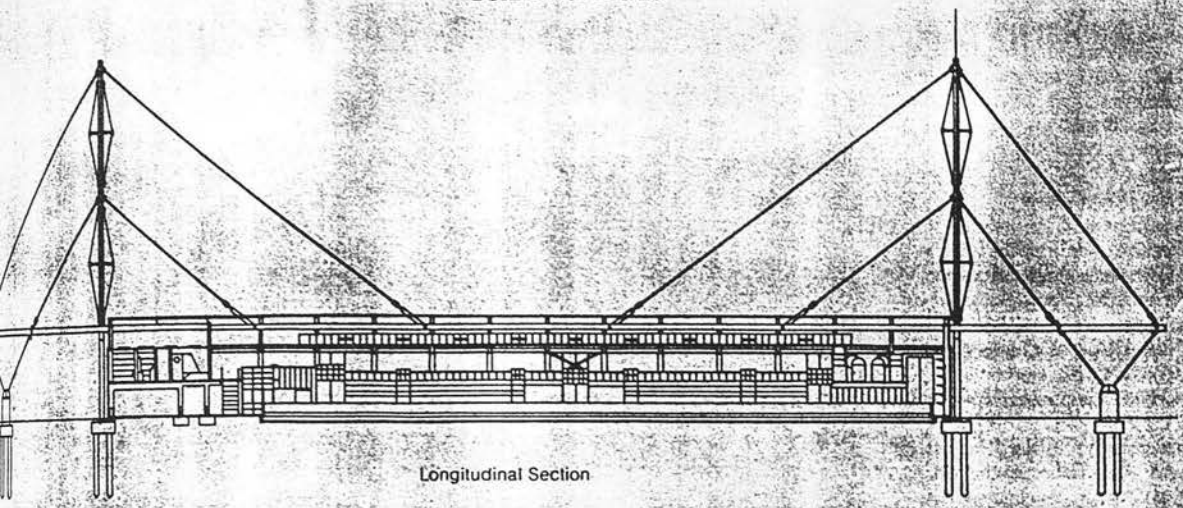
Ground Floor

1. 1 in 25 entrance ramp
2. entrance canopy
3. ticket kiosk
4. bridge
5. lessons desk
6. admin office
7. manager's office
8. shop
9. bar
10. raised bar viewing area
11. barstores
12. cleaner
13. disabled wc
14. male wc
15. female wc
16. conference room
17. bleacher seating
18. transformer enclosure
19. cooling tower
20. fixed seating
21. gallery

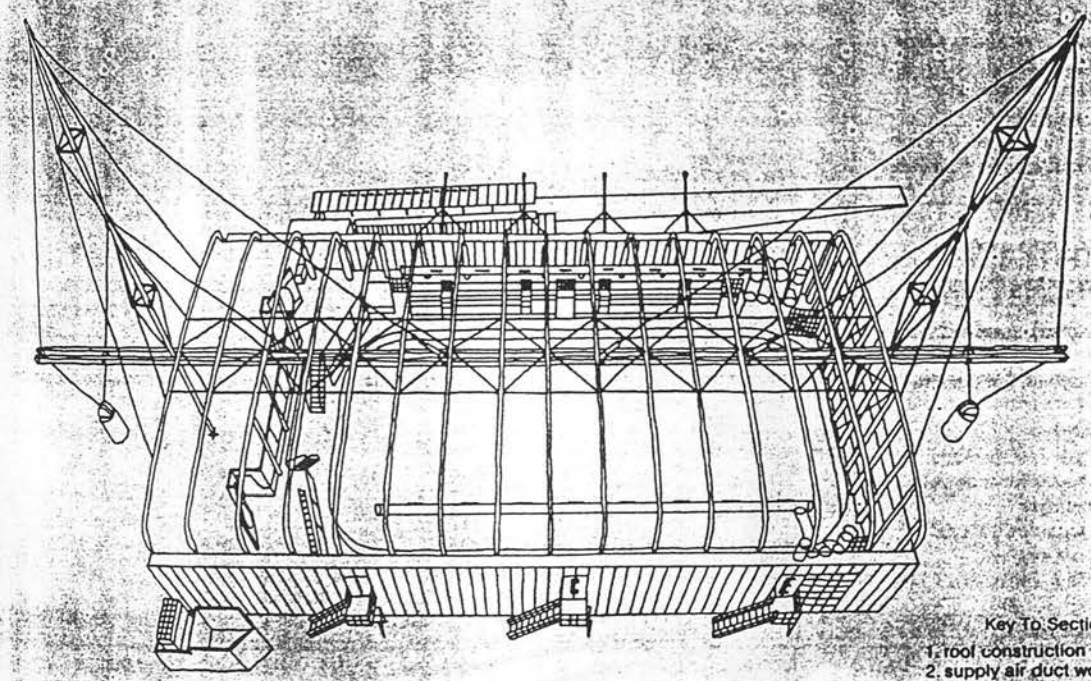
1. ice pit/ice machine garage
2. air plant
3. professional changing rooms
4. first aid
5. cafeteria
6. catering manager
7. food store
8. kitchen
9. servery
10. bottle store
11. cleaner
12. female wc
13. male wc
14. skate store
15. ice plant
16. water storage
17. l. v. switchroom
18. skate hire
19. tuck shop
20. staff changing rooms
21. boiler room
22. instructors
23. disco control/ judges, etc.
24. fixed seating
25. rink store



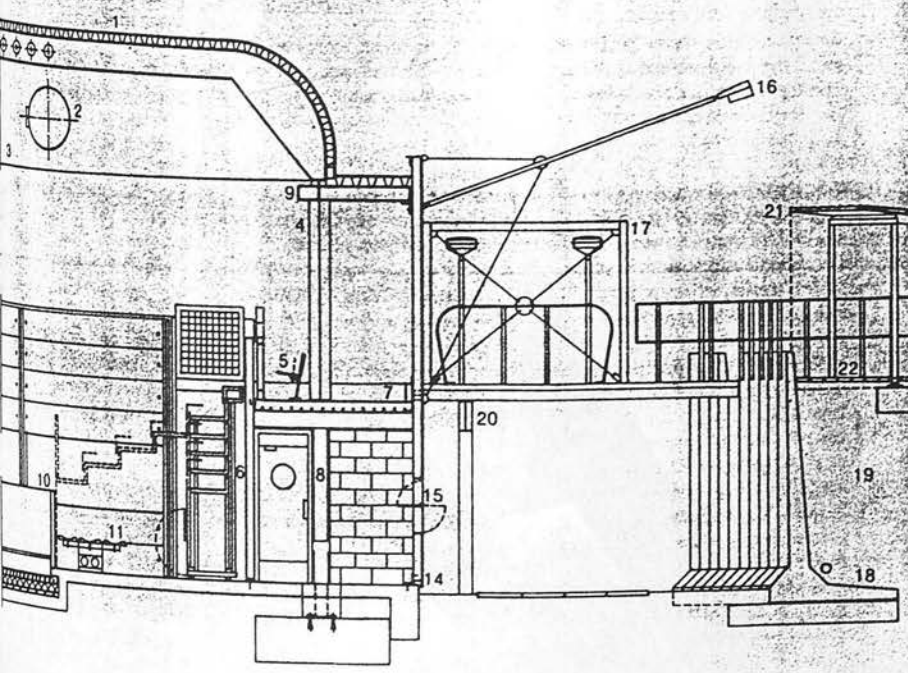
Cross Section



Longitudinal Section

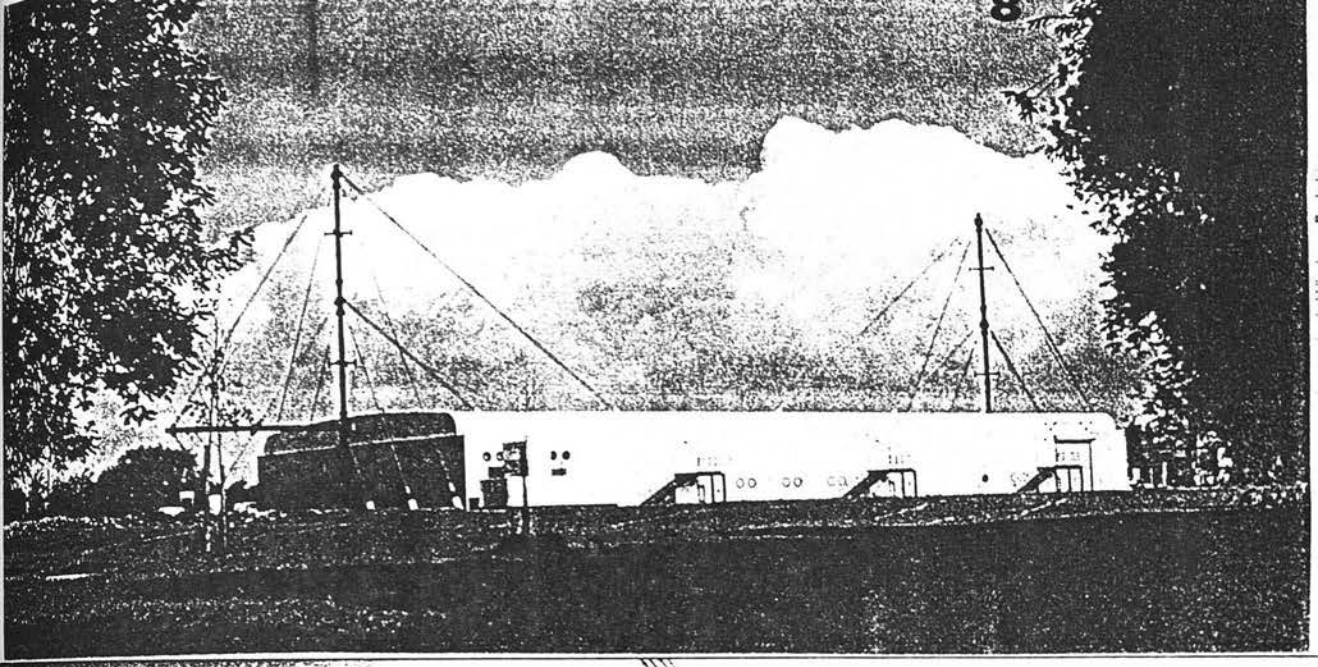


Aerial Perspective



Key To Section

1. roof construction
2. supply air duct work
3. bulkhead and fire-break over accommodation area
4. circular fire casings
5. proprietary fixed tip-up seats
6. autoclaved aerated concrete planks
7. reinforced concrete slab
8. steel cased mineral board fire casing
9. steel work supporting cladding
10. proprietary retractable bleacher seating
11. fixed tractor seats
12. rink barrier
13. ice pad
14. grc sandwich panels
15. aluminium porthole windows
16. external lighting
17. entrance kiosk
18. precast concrete retaining wall units
19. PFA fill material
20. entrance bridge frame
21. grp entrance canopy
22. interlocking concrete block paving



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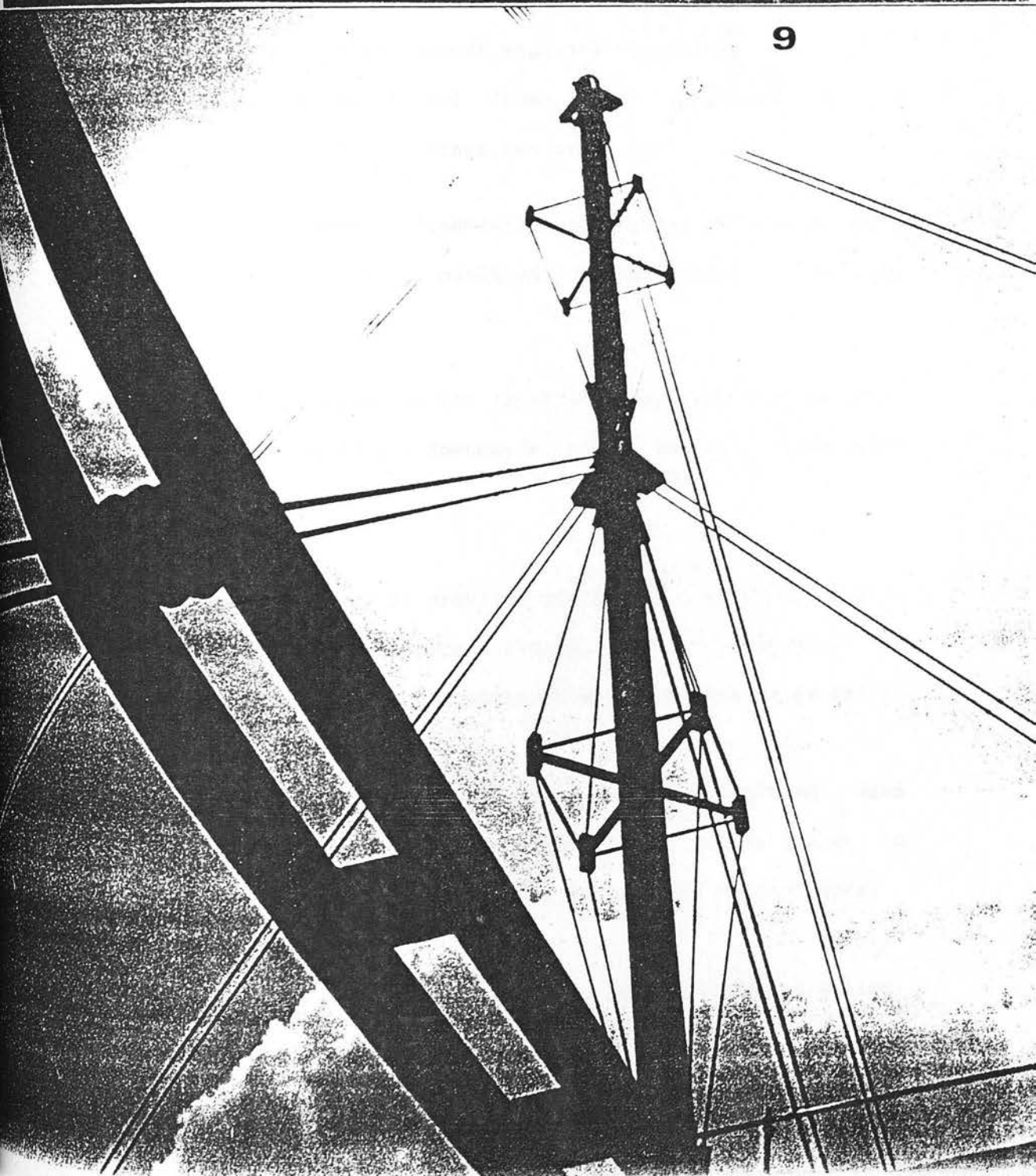


9

CASE STUDY No.19

LADKARN (HAULAGE) LTD
General Description of Structure to be dismantled and re-erected

Architect: Nicholas Grimshaw

OAP, Structural Engineers

Extracts from a report by Peter Higson, Trigram Associates

The building consists of three almost separate structures (Fig.1):

1 Office building-moment steel frame, not dependent for its stability on either of the other two structures.

2 External steel frame - dependent for internal stability on a connection to the office building, and a connection to the workshop box.

3 Workshop box - dependent on the external steel frame for vertical support (both upwards and downwards) at 12 internal suspension points.

Its lateral stability is provided partly by diagonally braced bays and partly by the office building. (The bracing alone is sufficient in the temporary conditions while the box is unclad.)

The office building is of conventional steel frame construction, with concrete floors which do not act compositely but do act as plates to distribute horizontal loads and equalize displacements at each level.

The workshop box is also of conventional beam and oclumn construction;

the beams support decking directly without the use of purlins. the decking is required to restrain the upper flange of the beams. Plan bracing is provided to distribute wind pressure to the braced bays and the office block. Beams are designed as continuous; columns (and brace members) are pin-ended. Site cladding spans horizontally between columns and is not required to contribute to the structural action of the box as a whole.

The external steelwork is essentially a pair of plane trusses, (11, 12) composed of tubular compression members and solid bar tension members. Apart from the wind loads it attracts and its own weight, the only force it resists are vertical ones from the workshop box roof. These can act in either direction, and two separate sets of tension members are therefore provided. The triangles formed by the main mast B1 or B2 and the two inclined members attached to it (S2 & S3) are exceptions; here the tubular members are resisting either tension or compression depending on whether net uplift conditions prevail. Also, the lower inner included truss 12 is subject to bending from the suspension point adjacent to the main mast (B1 or B2) (under both directions of loading).

With this exception, members are designed as pin ended. Tie bar members are assumed inactive when subject to compression. Tubular member are continuous through intermediate joints but are detailed with pin connections at the main intersections.

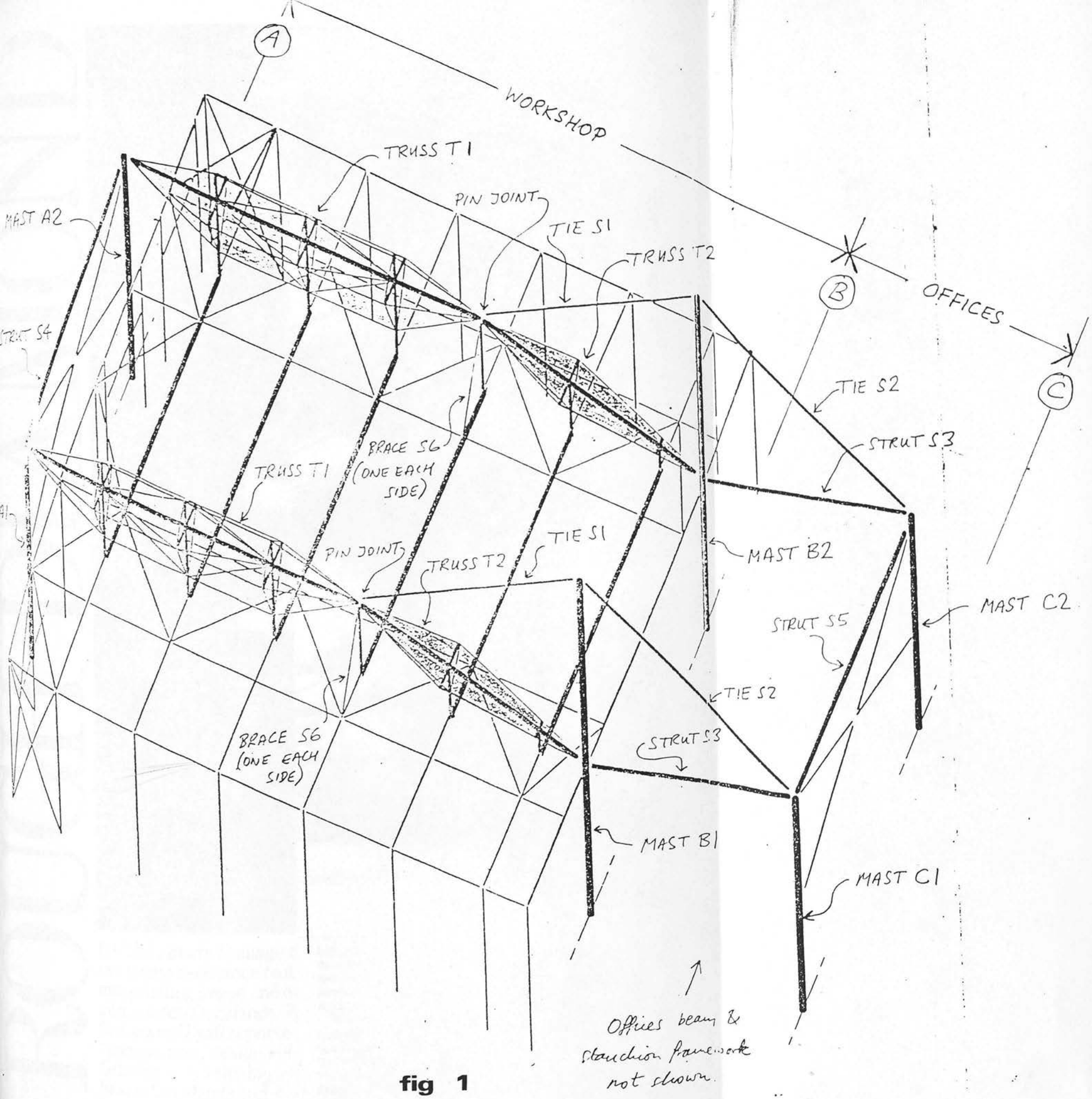
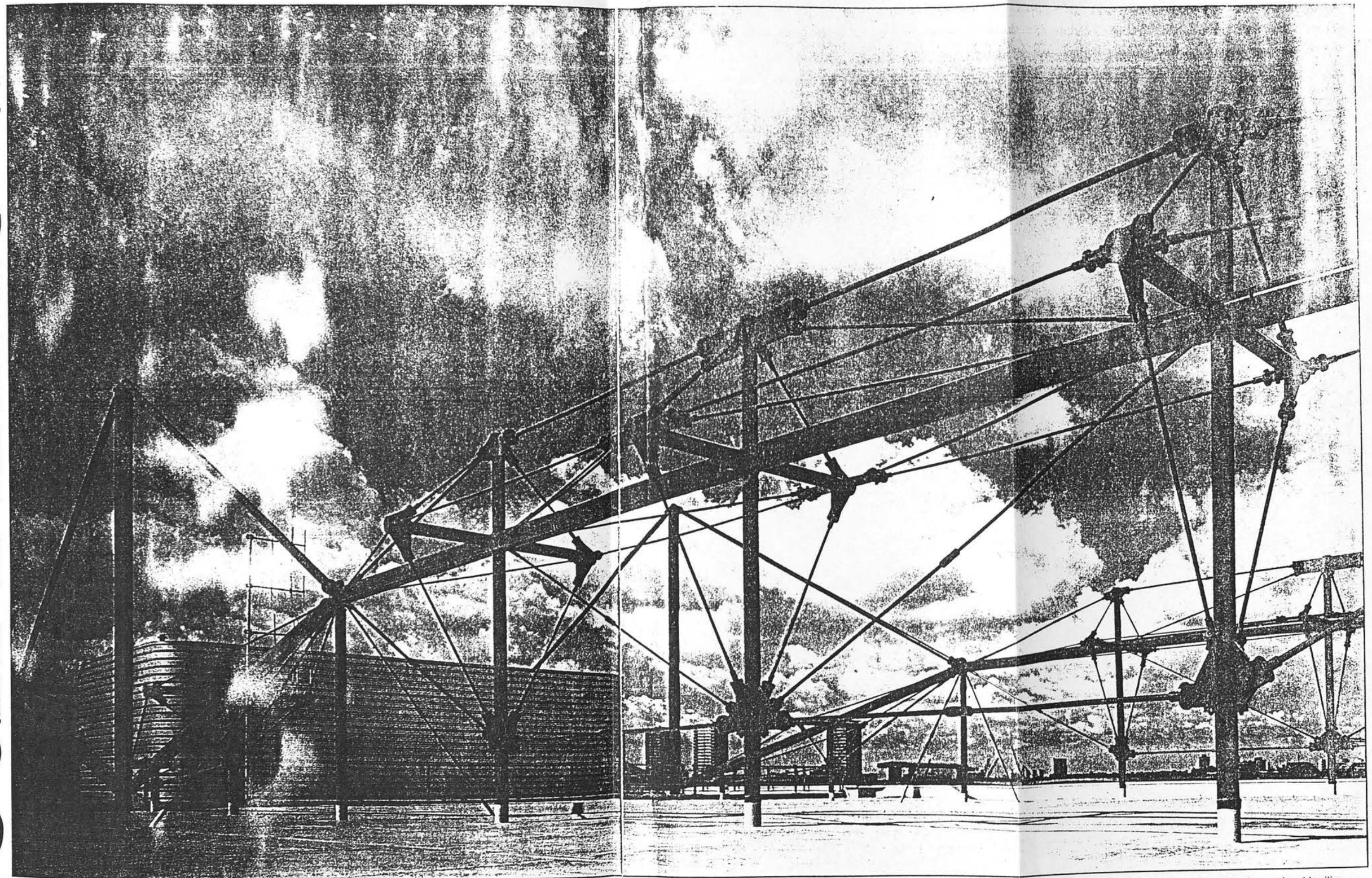

A
WORKSHOP
TRUSS T1
PIN JOINT
TIE S1
TRUSS T2
MAST A2
B
OFFICES
C
STRUT S4
TIE S2
STRUT S3
BRACE S6 (ONE EACH SIDE)
TRUSS T1
TIE S1
PIN JOINT
TRUSS T2
MAST B2
MAST C2
STRUT S5
TIE S2
BRACE S6 (ONE EACH SIDE)
STRUT S3
MAST B1
MAST C1
Offices beam & stanchion framework not shown.

fig 1

ROUGH DIAMOND



It took Ladkarn Haulage two years to overcome resistance to its muckshifting image and effect an entry into London Docklands. But, as *Sutherland Lyall* reports, its new headquarters, designed by Nicholas Grimshaw, is a shining example of the best of Docklands architecture.

Ladkarn Haulage's new headquarters on the Isle of Dogs is London Docklands' architectural gem. Designed by Hi-Tech architectural master-jewellers Nicholas Grimshaw and Partners, its gleaming round-cornered cladding and bright red external roof trusses shine out of the water just across from the end of Canary Wharf.

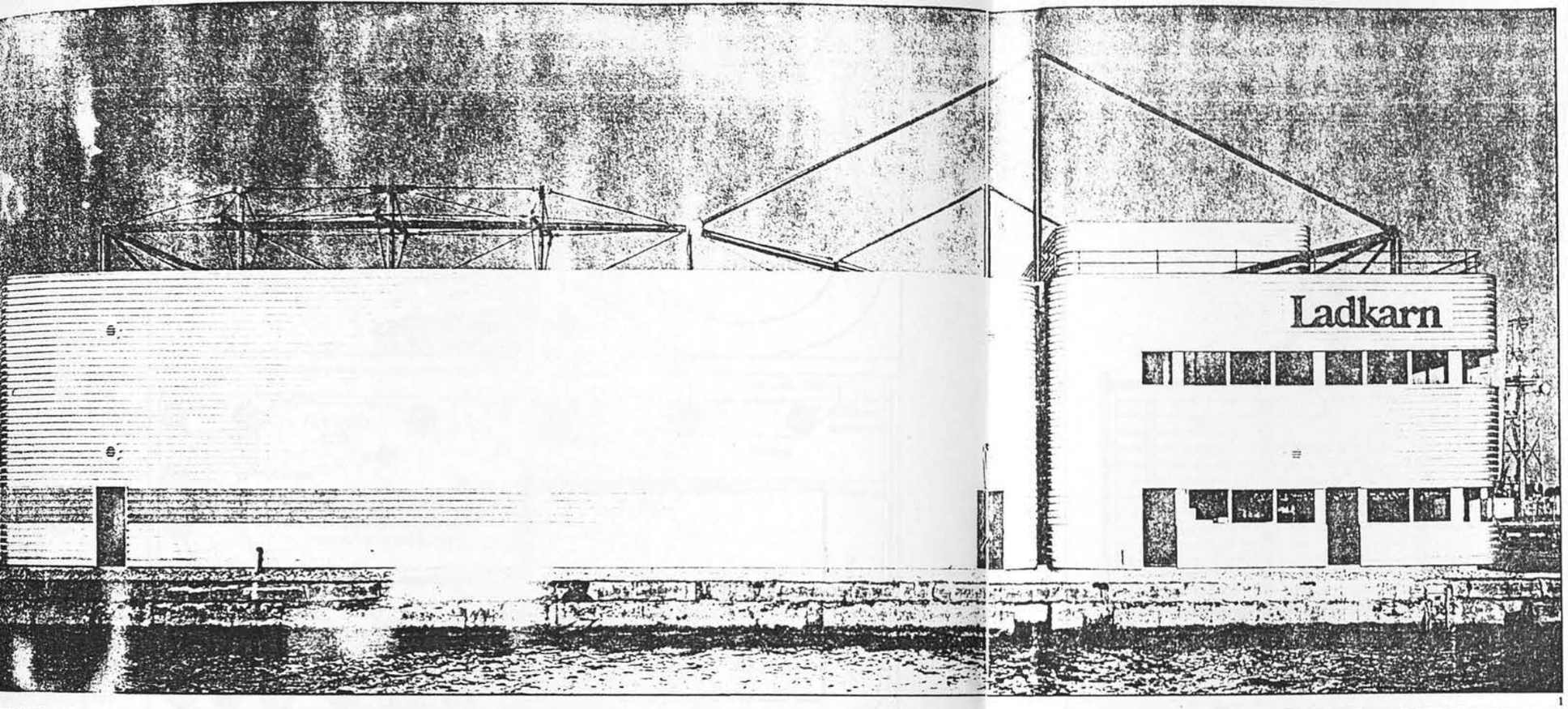
There is a certain irony in the fact that Docklands' current best bit of architecture came about only after considerable manoeuvrings in this allegedly planning-free zone. And in the fact that it may be there for only a brief time: it is in the path of the US-funded business megastructure currently emerging from the tentacles of the planning protest system.

Another nice irony is that this precise and gleaming building is home for a member of the construction industry whose sphere is the very dirty-fingernail business of muckshifting.

The silver building with its blue stripe is tantalisingly visible from many views at the top end of the Isle of Dogs, yet Ladkarn's headquarters is not an easy building to get to. It is on a dramatic promontory site, accessible only from an anonymous entrance at the north-east corner of the zone.

Whatever Calvinist structural engineers might say about the drama of its masted external roof trusses being more to do with visual design than engineering design, they have a comfortable visual consonance with the masts and rigging on the old sailing ships in the docks.

Set up in 1977 by Kevin Kennedy and Sean Burke in a couple of railway arches in Bermondsey, Ladkarn Haulage now has three main lines of business: hiring heavy earthmoving plant; disposal and delivery for big digging jobs; and civil engineering subcontracting. The firm had expanded into ►



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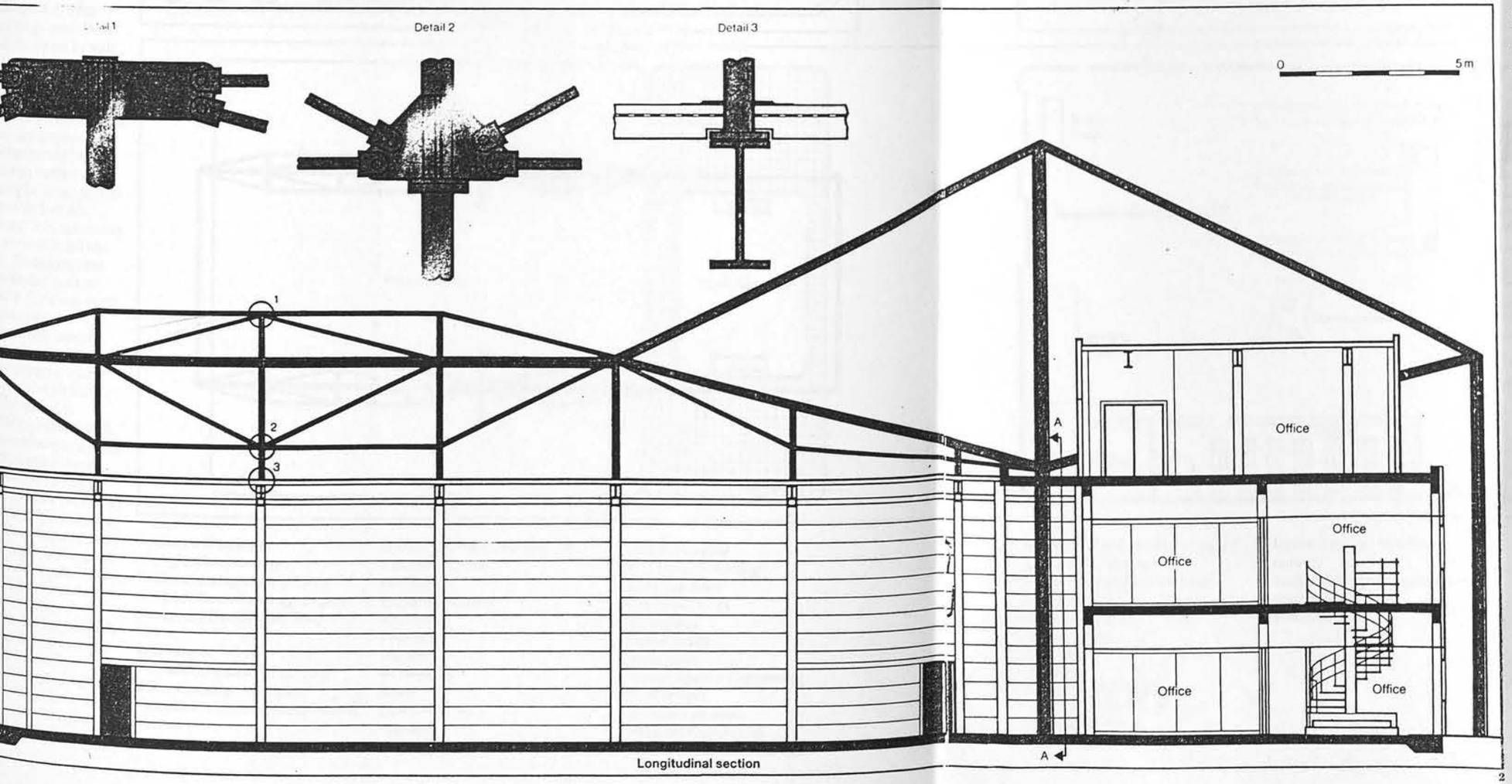

Longitudinal section

2

▶ temporary accommodation.

But several years ago, says Tony Green, Ladkarn's negotiator for the scheme, "we were getting on top of each other: we had to move. We had a lot of work in Docklands and in the City and looked set there for the next five years. The Docklands Enterprise Zone made sense geographically and, not least, from a tax relief point of view."

But the firm got the brush-off when it tried to talk to the London Docklands Development Corporation. LDDC plainly was not at all happy about letting loose a bunch of muckshifters among its more or less clean-fingernailed tenants. Says Tony Green: "I think they thought we would start doing things like fly tipping in the docks. There was a lot of suspicion."

But Ladkarn persisted, Green even wrote to his MP, they marshalled arguments to the effect that they and their staff were all local East Enders, and they persisted in the face of LDDC claims that all the Dockland sites were gone. Finally, the two got together. "When we met," says Green, "the LDDC said 'go away and show us what you want to do'. They also rather pointedly said that they were very strong on architecture."

Says Green: "In our business, we could have operated perfectly well from low quality premises. But we reckoned that if we were to move we had to look at a new building as an investment – and as a reflection of the whole company. So from the start we were looking to achieve something good. We would have been foolish if we had tried to build a shed. And now people come over here just to look at the building."

Ladkarn asked for a list of approved architects and went around to see what they had to offer. Some of the architects said they could not help. But not Nick Grimshaw.

Ladkarn had plenty of experience in doing the preliminary site work for new buildings. But like most building clients, the firm had never been involved in the stages before that – commissioning a new building. It was bemused to discover the way in which it traditionally happened and by the labyrinth of professional fee scales and percentages.

It was used to quoting a fixed price for a job and doing it. Very reasonably, it could see no particular sense in doing it differently when it was calling the shots. Green says: "Some of the professionals we saw tried to create the illusion that they weren't really interested in fees. We found that vagueness difficult to take.

"We were using our own cash plus bank finance. Nick was sensible and understood that we needed to know how much he would cost right at the beginning. And he gave us a fixed price."

Grimshaw associate David Harriss gave Ladkarn an outline design for a flexible building: offices with big repair workshop space behind and hardstanding beyond that. It was a simple design with the possiblity of converting workshop area into warehousing or factory space, or into more offices. Says Green: "It was a specialist building. But it would require very little work to change it around. That made it a good investment."

The firm took Grimshaw's outline model back to LDDC senior architect Chris Attwood. Hardly surprisingly, he liked it and the LDDC effectively had to find a site. But after offering and then withdrawing three sites, the corporation sent Ladkarn off to the Port of London Authority which owned a finger of land in the north-east corner of the ▶

1,2, Elevation. Details are of plate connections for the steel frame, custom made by Arup for each type of joint.

The office and workshop facility consists of three almost separate structures. The 653 m² office building is of conventional steel-frame construction with concrete floors which act as plates to distribute horizontal loads. It is not dependent for its stability on either of the other two structures.

The 899 m² workshop is of conventional beam and column construction. It is dependent on the external steel frame for vertical support at 12 internal suspension points, lateral stability being provided partly by diagonally braced bays and partly by the office building. The beams support decking directly without the use of purlins. Beams are designed as continuous; columns are pin-ended. Side cladding spans horizontally between columns.

The external steel frame is dependent for stability on connections to the office building and workshop. It is essentially a pair of plane frames, composed of tubular compression members. Tubular members are continuous through intermediate joints, but are detailed with pin connections at main intersections. The main masts are propped at second floor level by a connection to the office building. Fore and mizzen masts cantilever from their bases and are connected by a hollow section truss at high level.

Wharf. "The PLA was
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the rigmarole about
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down-side was that
had to build the access
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ked. Explains Green:
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we would have a word
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basis.
m's David Poole, who
project manager, says:
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to us. We got by with
sense and the expertise
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the LDDC made it a bit
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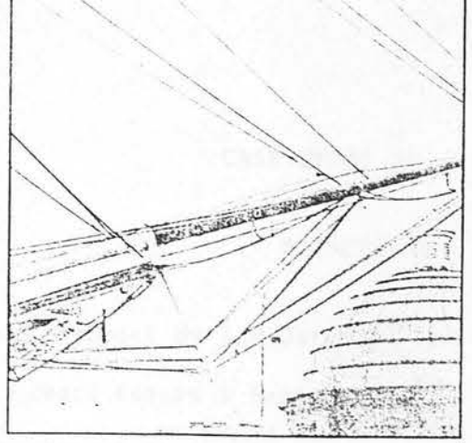

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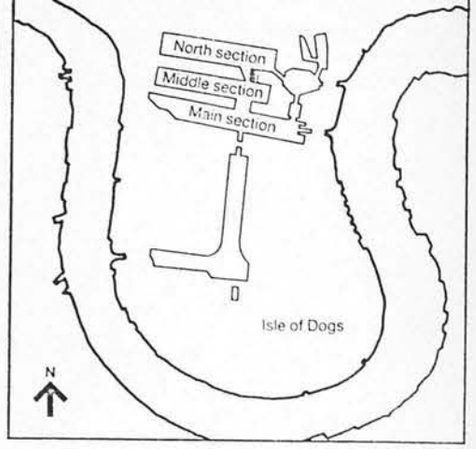

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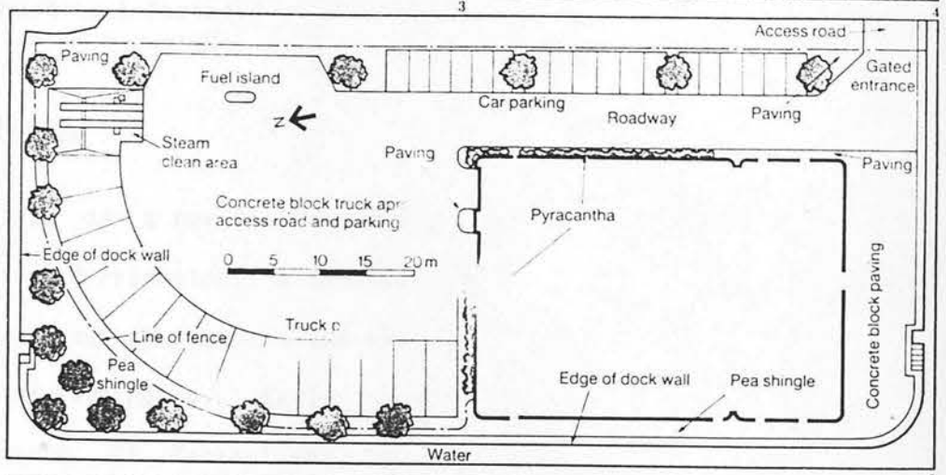

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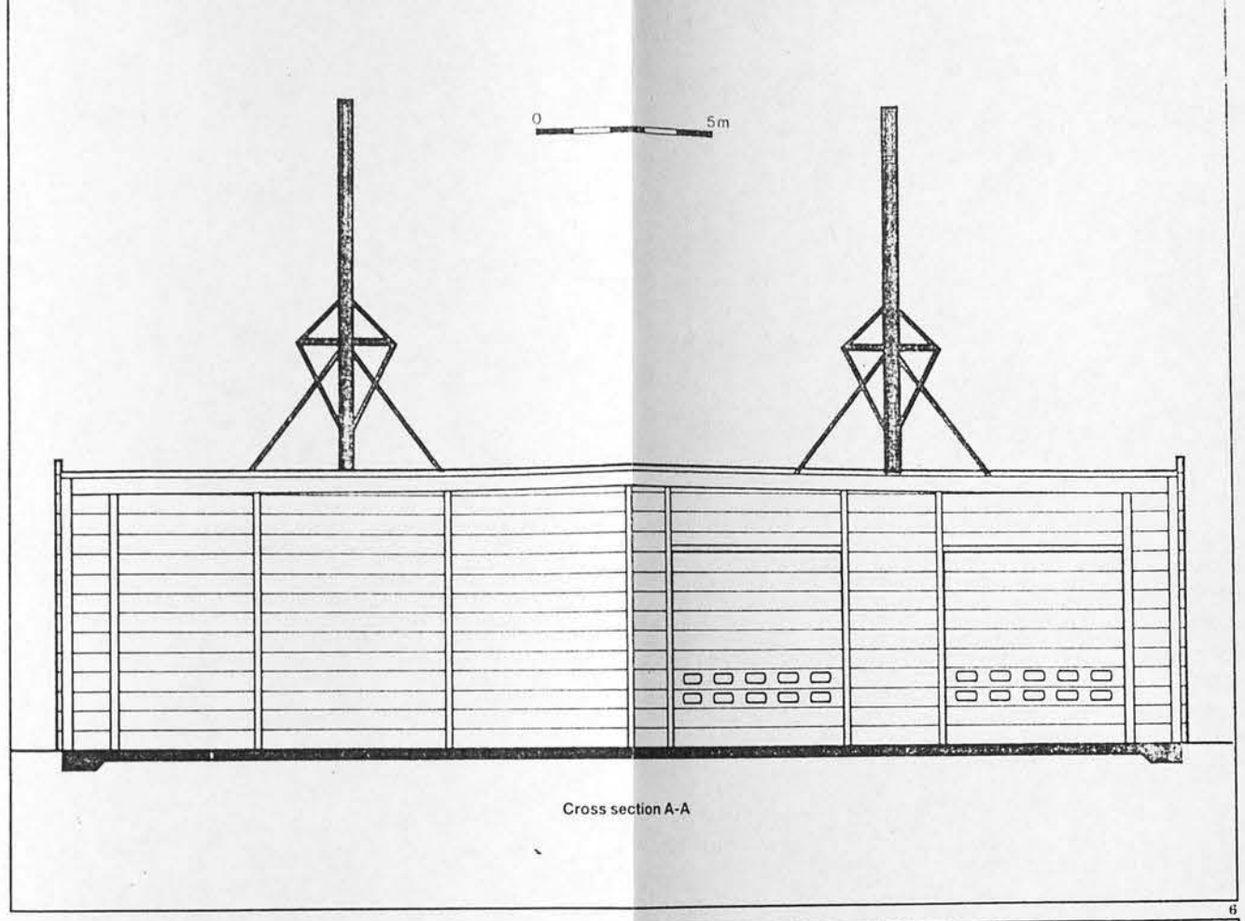
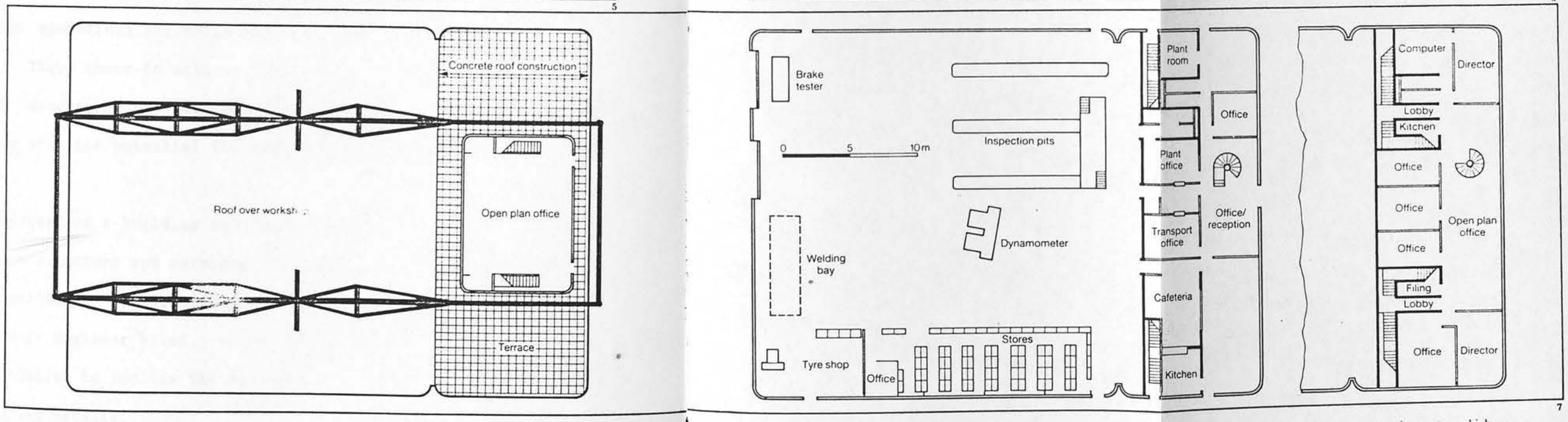


Cross section A-A

6



7

Ladkarn Haulage
client/general contractor
Ladkarn (Haulage) Ltd: project director – Kevin Kennedy; project managers – David Poole, Tony Green
architects
Nicholas Grimshaw & Partners: partner in charge – Nicholas Grimshaw; project architect – David Harriss
structural/service engineers
Ove Arup & Partners: director – Richard Haryott
electrical
Ladkarn Electrical
mechanical
Powermart
shutters
R O'Rourke
piling
Dowsett Prepakt
steelwork
Painter Bros
cladding/roofing
R M Douglas Roofing
workshop doors
Crawford Doors
fire shutters
Bolton Brady
spiral stairs
Donald Mackay Engineering (Cambridge)
secondary steelwork
Balguard Engineering
partitions/suspended ceilings
Interfinish Partitions International
workshop equipment
Industrial Equipment Services
smoke vents
Crittall Tectonic
fire protection
Fireboard
suppliers
Gavle UK (cladding);
Rockwool, Dow Chemical Co (thermal insulation);
Sarna UK (roof membrane);
Hyatol Windows (windows)
contract
Ladkarn's own management contract
contract period
8 months

3, The bowsprit of the Cutty Sark reveals the inspiration for the steel-frame design.
4, The building sits on a promontory on the Isle of Dogs across the water from the end of Canary Wharf.
5, Site area is 0.45 hectares, total building area is 1552 m^2 – a site cover of 28%.
6, Section through building in front of main masts looking into the workshop area. The piling is deeper than the masts are high.
7, Plan at roof level *(left)*. The workshop has profiled steel roof decking with Rockwool insulation and Sarnafil pvc roof membrane. Offices have concrete roof slabs finished with paving slabs to form a terrace. Plan at ground level *(middle)* showing offices and pits for lorries. Workshop has reinforced concrete floor slab. Plan of first floor *(right)* shows office layout.

CASE STUDY No. 20

PATSCENTRE

Extracts from a report by Ian Gardner, OAP

Architect: Richard Rogers & Partners

Engineer: Ove Arup & Partners

1.0 Introduction

Patscentre is a new research facility for PA Technology on the outskirts of Princeton, a university town which is close to the American eastern seaboard and approximately mid-way between New York and Washington. Early in 1982, management and technology consultants, PA Technology, were seeking to rationalise their American operations and while doing so gain a more distinctive image. They chose to achieve this by appointing the architect Richard Rogers to provide them with a laboratory and corporate facility with the potential for easy growth.

The success of a building such as Patscenter with its highly expressed structure and services is very dependent on the quality and consistency of detailing. This was recognised in the structural engineer brief, which was extended beyond the basic scheme design to include the development in principle of the key engineering details.

2.0 Building layout and concept

The nature of the research projects which PA Technology are

appointed to carry out varies considerably. Their work involves both desk-based studies and practical experimentation; both office areas and laboratories are therefore required. However, the mix of spaces and the layout of rooms needs to be adaptable to enable different combinations of research commissions to be undertaken.

Other less flexible areas are also required. Early during the briefing stage, specific areas were scheduled for central administration office, computing library, reception and conference facilities.

This combination of flexible research space and less flexible central facilities, together with the client's expressed desire that the building form should be readily extendable , determined the concept for the building's internal planning. The central facilities are located in a 9.0 m wide spine which also duplicates as the principal circulation zone. On either side of this spine two large single storey enclosures, each 72 m long x 22.5 m wide, provide the research space. To achieve the required flexibility these research areas are organised on a 9.0 m x 4.5 m planning grid and are column-free. All vertical structure is contained within the central spine, or is external to the building envelope.

The desired layouts of offices and laboratories have been formed by erecting free-standing demountable enclosures within the completed building. These subdivide the two large research spaces into combinations of rooms, generally using the discipline

of the planning grid.

The resulting building plan is essentially linear with a dominant symmetry about the spine (Fig. 1). From the outset the architect was keen that this should be reflected in the building form. Richard Rogers felt that the building should be perceived as a series of slices, each representing a one bay module. Further slices could thus be added at a later date without impairing the concept and visual integrity of the building.

The structure is externally expressed to achieve the column-free research enclosures and equally important, to provide the main architectural theme for the building. The large single-storey building, with its general roof level only 4.5 m above ground level, is enlivened by the deliberately dramatic steelwork frame (Fig. 2). Major services plant is suspended above the central spine keeping it clear of the main building envelope. The building services also, therefore, contribute to the architectural image (Figs 3 and 4 A, B)

3.0 Structural frame

3.1 The structure comprises a row of nine identical frames spaced at 9.0 m intervals along the building length. Each frame has a stiff 7.5 m wide portal within the central spine, above which extends a rigid 15 m high bipod mast. Inclined tension members splay out symmetrically from the top of this mast to provide mid-span support for the main roof beams over the research enclosures.

Whilst the geometry of the bipod masts and their supporting portal was established early in the project, it took longer for the geometry of the tension system to be developed (Fig. 5).

3.2 Development of Structure

(Effects of geometrical arrangement of structure on efficiency)

The engineers requested that the suspension system must incorporate a truss to resist wind uplift, and thus avoid the inefficiency of having to ballast the roof down with sufficient dead weight to maintain always a net downwards loading. The initial scheme (Fig 5a) had twinned inclined hangers on each side of the mast connecting to the roof system at 1/3 span points. This was originally based on clear roof spans of 27 m. However, for the 24 m roof spans which emerged from the internal planning, this arrangement was considered over-elaborate and the outer hangers were found to be making very little contribution to the roof support.

A hanger with an inclination to the horizontal of less than about 30 degrees does little to prevent vertical deflection of the roof and, at such a small inclination, the hanger tends to sag visibly under its own weight. Its axial stiffness, initially at any rate, is therefore that of a shallow catenary rather than a direct tension member. Also, with a central mast arrangement such as the Patscenter building, the horizontal component arising from the inclined hangers is resolved at roof level by the primary roof beams carrying compression forces back to the stiff spine. The outer hangers of small inclination tended to put an unacceptably high compression component into the main roof beams.

In scaling down the roof suspension system to one which seemed more appropriate for the 24 m spans the outer hangers were removed (Fig 5b). However, the very clear and symmetrical arrangement for the wind uplift truss over each roof span, which was liked by the architects at this stage, caused further structural problems.

This new geometry was approved by first realising that, even with pinned joints at the main roof beam suspension points, the system was not a mechanism - it would have been absurd to have the whole stayed-roof system dependent on the bending stiffness of the primary beams. Assessing whether a structure containing a high proportion of tension-only members is statistically determinate is not always immediately obvious. The engineers chose the simple graphical method of plotting the loci of node displacements for rigid sub-assemblies and then assessing whether the omitted tension members would have had to increase in length to follow these loci (Fig. 6). If a displacement locus was possible with all omitted tension members either maintaining or reducing their original length, then the system would clearly be a mechanism.

Although the system was not a mechanism it proved highly inefficient. To prevent lateral displacement of note C much of the tension in member AC had to continue as a tension in member CF. This both reduced the vertical upward reaction available at node C to take the roof loadings and put large compression loads into the roof beam at node F. Also member BC could only be prevented from going slack by enormously pre-tensioning member CF

(Fig 7a). Again since further tension in member CF resulted in increased compression in the roof beam, this would have been counter-productive.

Inclining the central triangle CDE simultaneously solved all of these problems (Fig. 7b) and provided the final as-built geometry (Fig. 5c). The main hanger became more steeply inclined, thus benefically reducing its horizontal component. The resultant line of action of members CD and CE fell within the enclosed triangle ABC, automatically keeping member BC taut. It also became possible to minimize the bending moments induced in the primary roof beam by initially setting the resultant of CD and CE just within the line of AC. Fine tuning is then achieved by controlling tension in member CF to pull the resultant further round.

The uplift wind truss, although possibly less clearly stated, is still provided by this final geometry. Members CD and CE are required to act as compression struts when there is a net uplift loading, but their lengths have not been greatly increased relative to the previous geometry. They therefore remain as sufficiently slender tubes for the overall tensile effect of the suspension system to prevail. In fact, the tensile effect tends to be enhanced by the asymmetry of triangle CDE with the system looking more taut, and the various members all appearing to have been drawn upwards towards the masthead.

4.0 Mast stability

Longitudinal stability of the row of nine bipod masts is

provided indirectly by making use of the suspended services plant platforms and their support hangers. This has enabled the structure, when viewed from the side elevation, to appear relatively simple and uncluttered. The masts project upwards at 9.0 m centres independently of one another, conveying the image of the building being segmental with a bay-by-bay add-on flexibility (Fig. 8a).

Out of plane loadings on the masts and suspension systems are transmitted down to the main roof level via the structural chassis of the services platforms. All horizontal forces associated with the vertical support systems are then resolved at roof level and transferred to ground level through the combination of central portals and diagonal bracing at the ends and sides of the building.

This solution for the longitudinal stability was not immediately arrived at. The engineers started by proposing diagonal cross-bracing between the masts along the building length, but the architects were keen to preserve the planarity of the main roof suspension systems. They would not accept longitudinal members connecting to the bipod masts; at least not above the services plant where they could be seen. The engineers did not want to introduce restraint at low level to the bipod masts since this would introduce bending stresses and compromise their behaviour as simple axial struts.

An apparently unrelated design decision solved this problem. The architects decided to light naturally the central spine with a

skylight and to do this the roof-mounted services were raised clear of the roof onto services platforms, continuous along the length of the building. It was obvious that these platforms should be suspended from the bipod masts since they further justified the need for the masts. This was done using hangers from the mastheads. Cross-bracing the services platforms down to the main roof level would prevent them from displacing longitudinally, and it was realised that they could thus be used to indirectly stabilise the masts.

If the platform support hangers were in the same vertical plane as the bipod mast a stable equilibrium system resulted (Fig. 9a). As the mast rotated under the influence of out of plane loading a restoring force was mobilized by the change in geometry of the system. Although the system had no initial stiffness, the rate of gain of lateral stiffness was rapid. The engineers decided to anticipate this gain and by presetting the platform support hangers at relatively slender angles to the bipod mast, obtained the final mast geometry (Figs, 9b and 10).

Interestingly, therefore, whilst the services platforms are held in place by the bipod masts, it is these same platforms which prevent the masts from toppling over. Without the building services plant there would be no requirement for the platforms and hence no mast stability system. Thus the building services help to justify the structure and vice versa. Also at no point do the horizontal platforms connect directly to the bipod members. This is emphasized in the completed building by the different colour paint finishes, to maintain the visual clarity of the simple bipod masts transmitting the building's weight down

towards the ground.

5.0 The implications of form and structural arrangement on Services distribution

Patscenter is a highly serviced building of approximately 4000 sq m floor area. The occupied space is entirely on the one ground floor level. At scheme stage the architects were asked to allow for a comprehensive range of services to be incorporated. The main task, therefore, was to establish the method of horizontal services distribution.

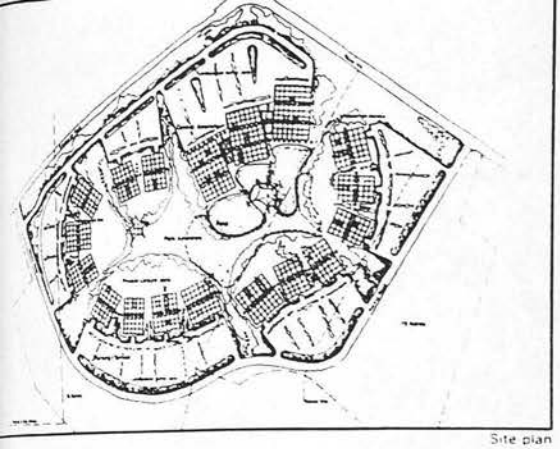
The planning concept of the central spine provides an ideal route for primary services distribution and enables major plant to be located centrally without impeding the use of the building. Mechanically and electrical plant are located at ground floor level adjacent to the spine. Air handling and condenser plant are located on the platforms above the spine. The two research enclosures are equally sized and on either side of the spine, so both plant and primary distribution are effectively place at the centre of services load and can take full benefit from any load diversities. The bulky primary air ductwork is external to the building envelope whilst the primary electrical and piped services are at high level within the spine (Fig. 8b). The absence of specialist research activities within the spine ensures primary services are readily accessible for maintenance.

Lateral feeders running internally at high level provided the secondary distribution to the research enclosures. In these areas an integrated zoning strategy has been evolved for the

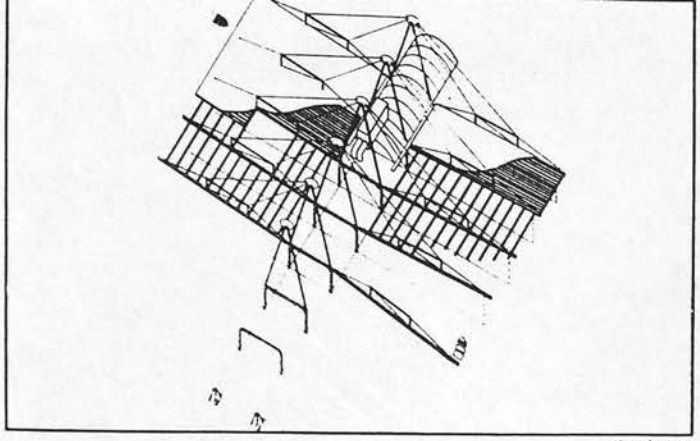
services and building (Fig 11). Such a strategy determines a fixed zone or route for each service or band of services, with none being allowed into anothe's zone. It ensures that all services will in fact fit into the building, standardizes installation details, and greatly assists maintenance.

The very rigid discipline which this zoning strategy imposes might be thought to inhibit adaptability. However, the reverse is true. By applying the same zoning strategy throughout the building a space is allocated for every service in each planning module. This is the case whether or not all of the services are initially installed in every module. Therefore, if the usage of an area is changed and in consequence requires a previously omitted service to be installed, then this is always possible because a distribution philosophy and route already exists for the service.

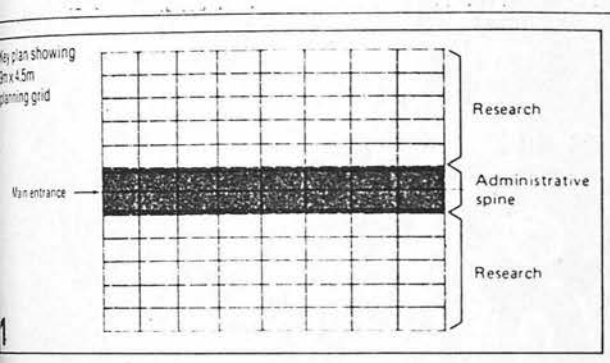
One further advantage of the zoning strategy is that the rigid discipline makes it possible for the services to be expressed. It was known from the start that the engineering services were to contribute to the architectural image of the building, both inside and out. Without precise control over the locations of all services it was unlikely that they would be visually acceptable.



Site plan



Kit of parts



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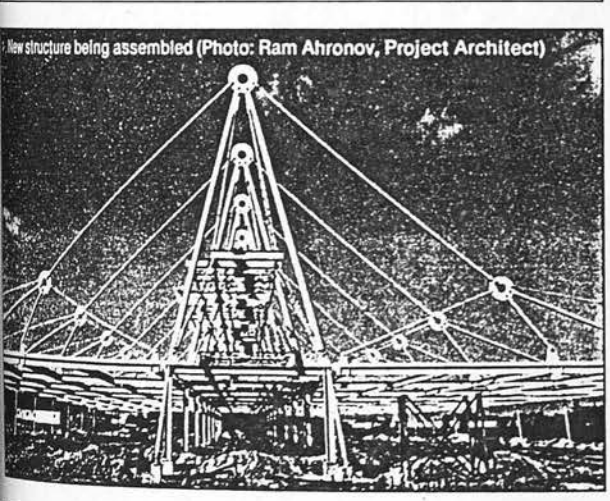
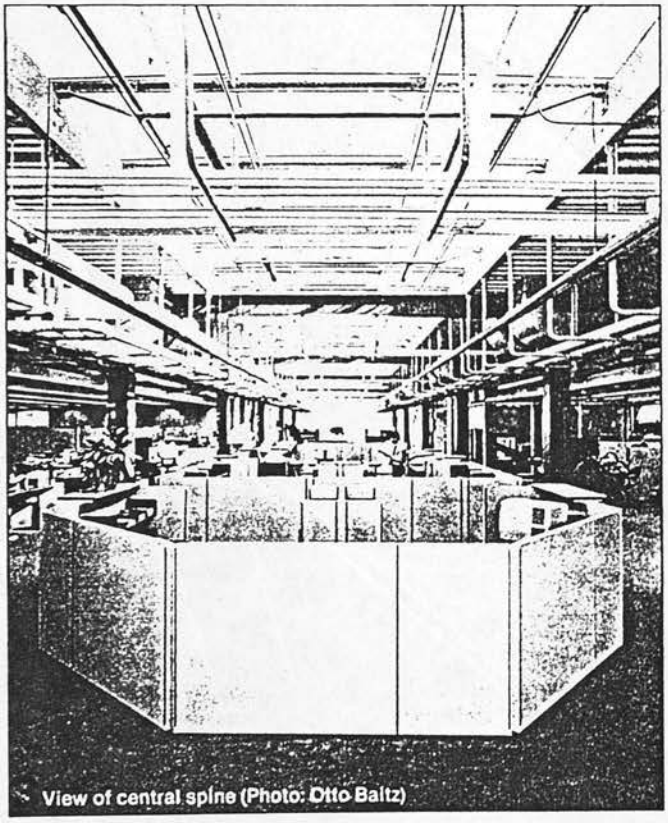
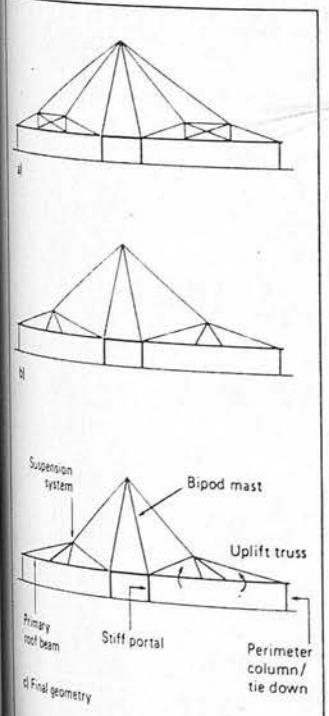


New structure being assembled (Photo: Ram Ahronov, Project Architect)

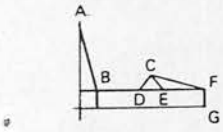


View of central spine (Photo: Otto Baltz)

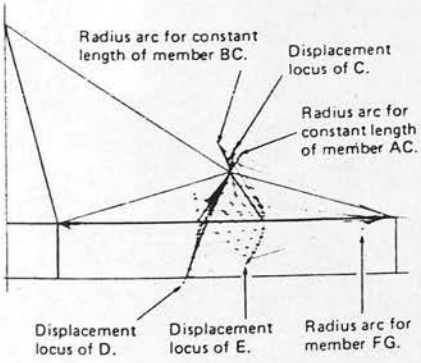


Evolution of structural frame

5

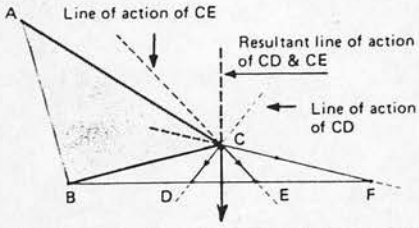


The members shown above remain fixed in length in the displacement locus shown below.

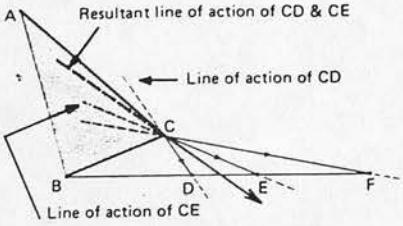


Plot of node displacements for one of the sub-assemblies considered. Shaded areas show that either member AC or member BC would need to increase in length.

6

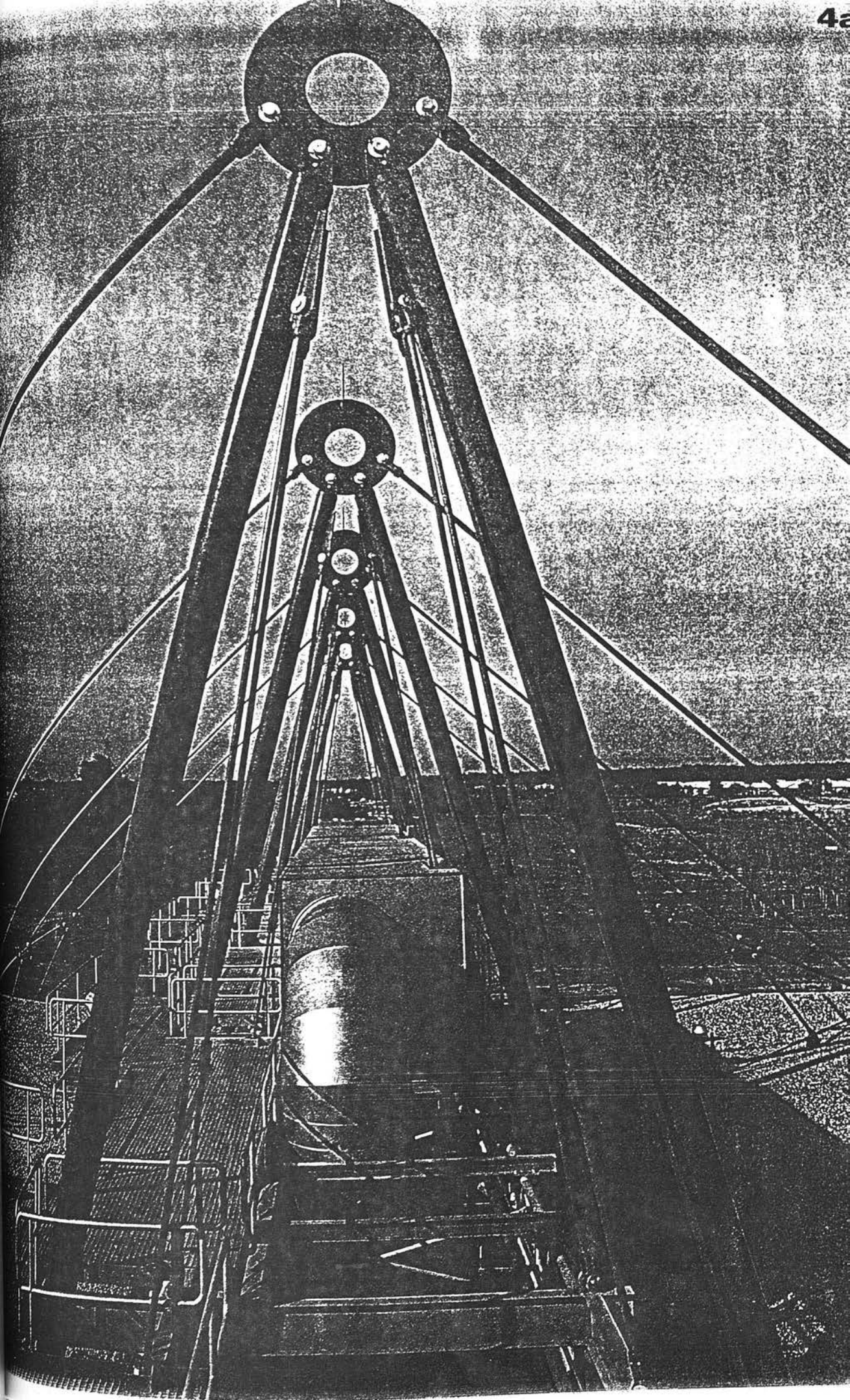


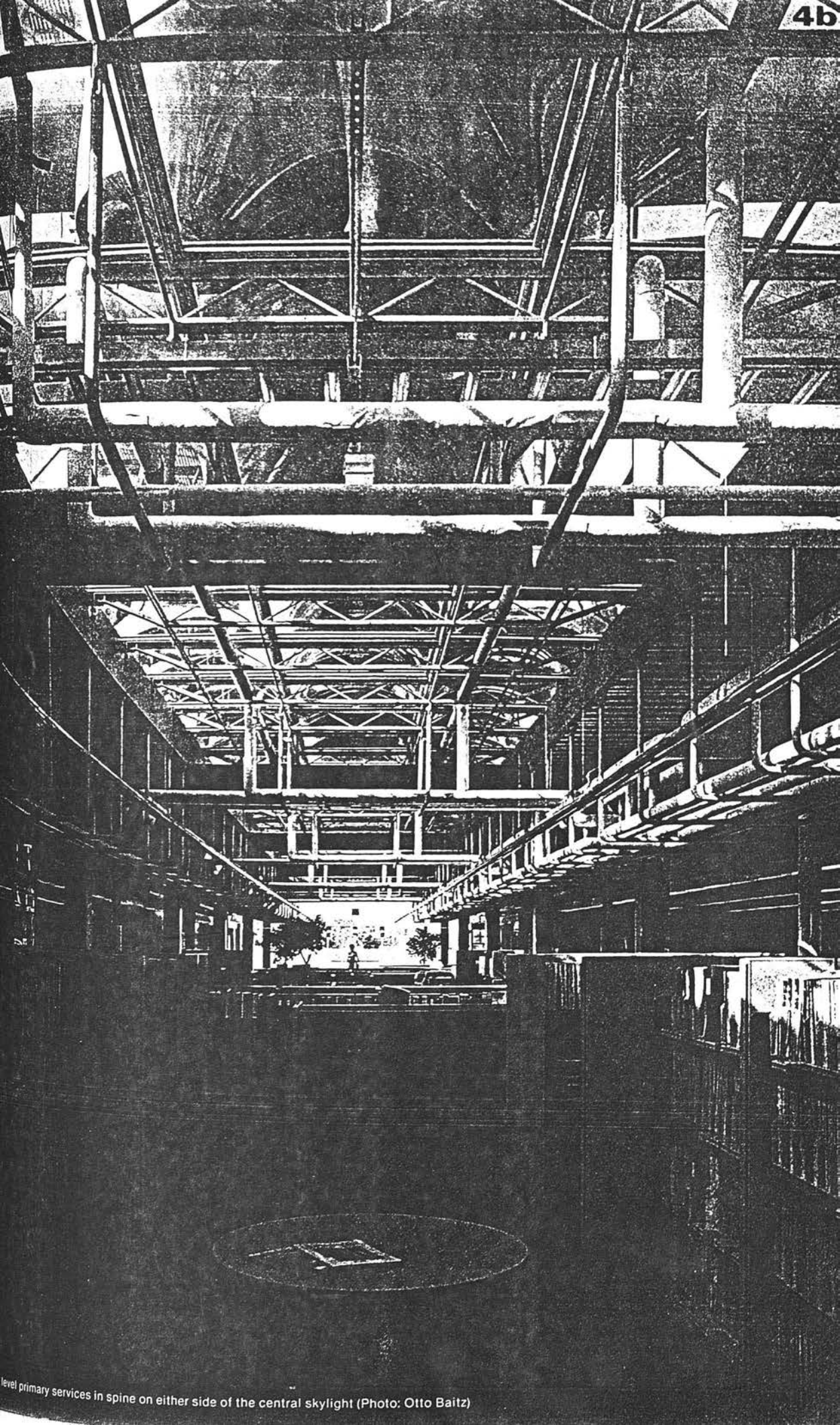
a) The lines of action of load-carrying members CD and CE are both outside the triangle enclosed by members AC and BC. The resultant line of action must consequently also be outside the enclosed triangle. It is therefore not possible for member BC to act in tension under the action of members CD and CE.



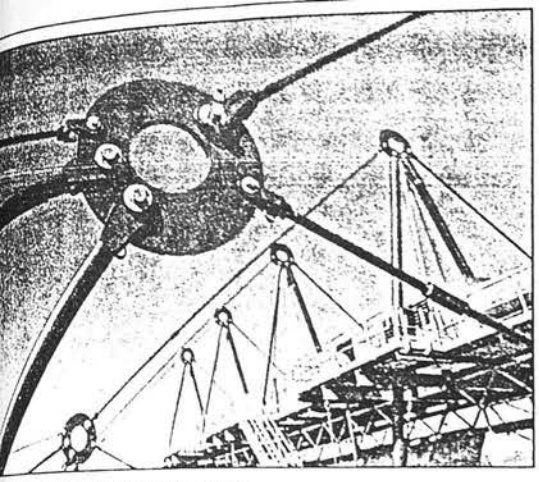
b) Although the lines of action of load-carrying members CD and CE are not both inside the triangle enclosed by members AC and BC, their resultant line of action is inside the enclosed triangle. Members AC and BC are therefore both tension members.

7

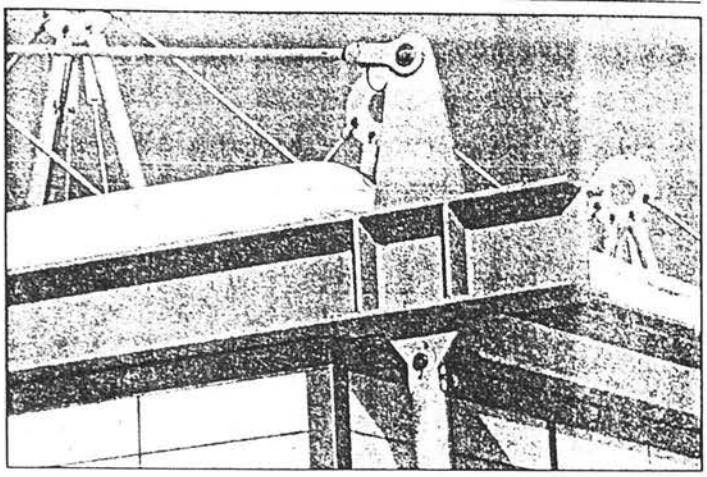




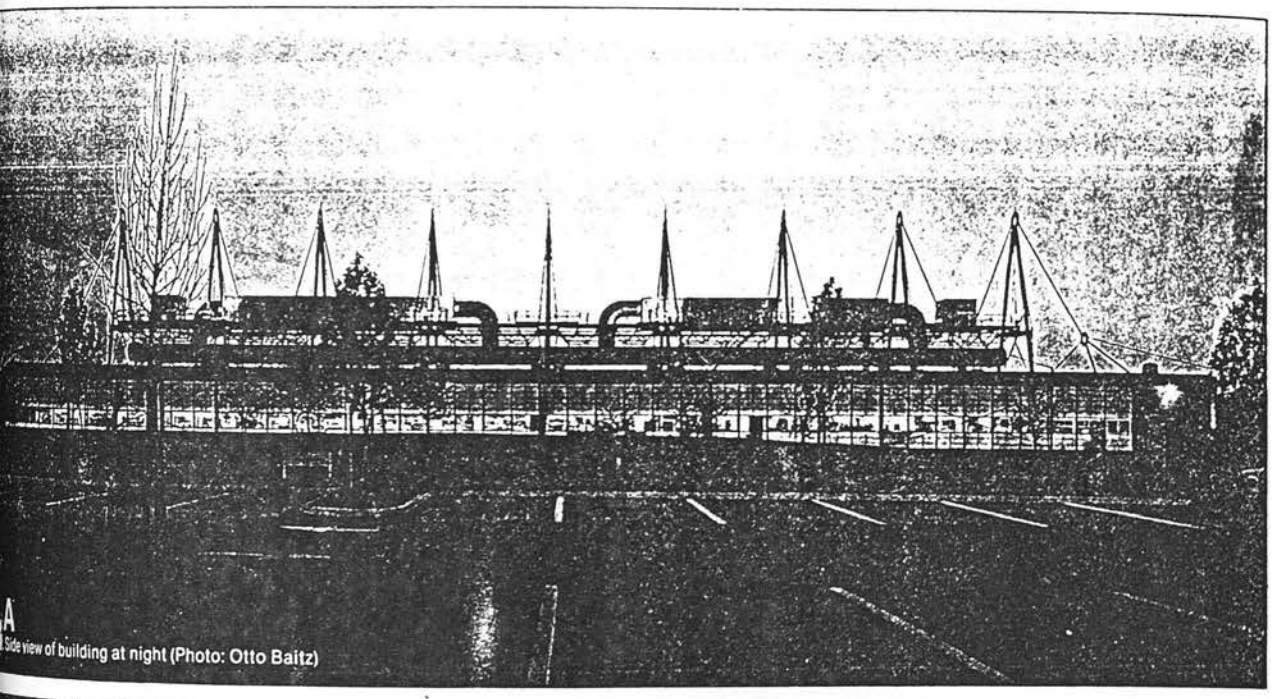
level primary services in spine on either side of the central skylight (Photo: Otto Baitz)



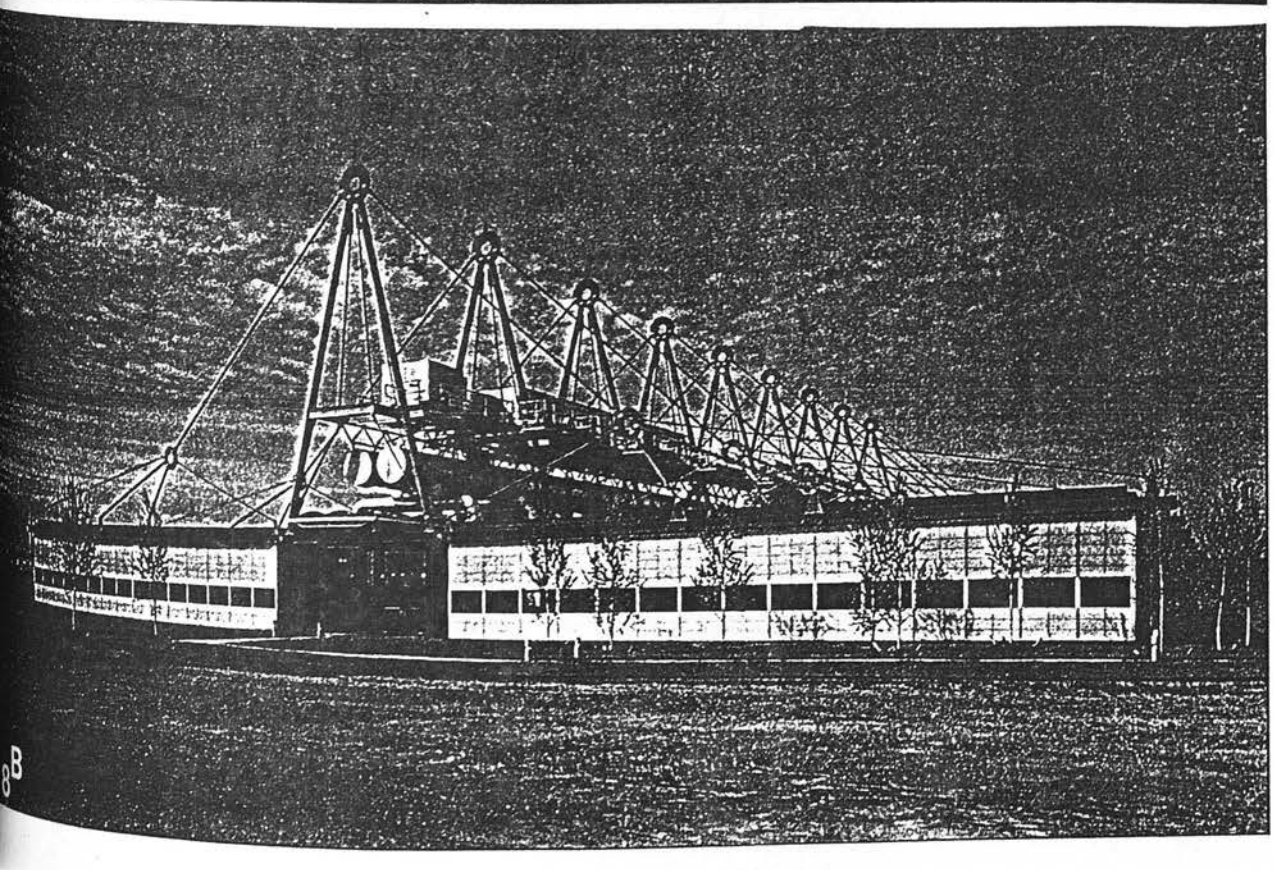
nular node plate of suspension system,
ed tapes for continuity of lightning
ction can just be seen (Photo: Barry Dunnage)



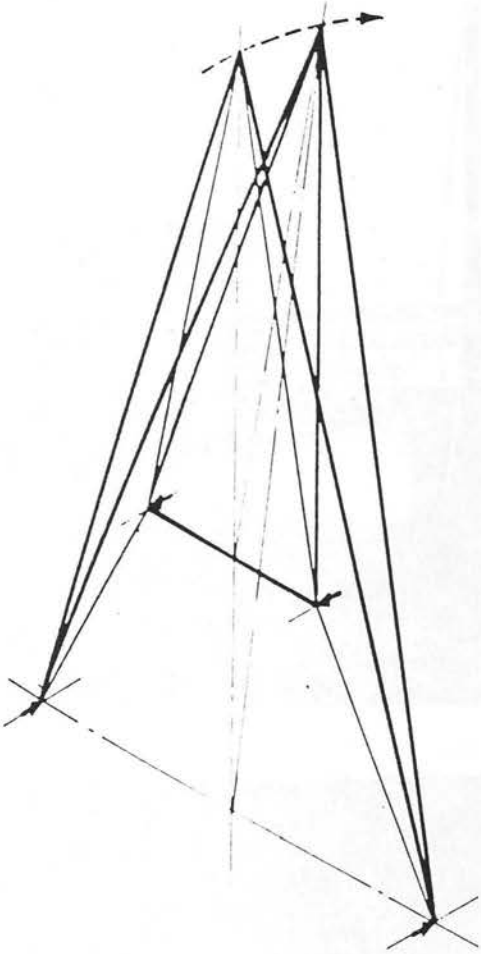
External view of corner during construction,
again showing looped tapes for continuity of lightning
protection (Photo: Ram Ahronov)

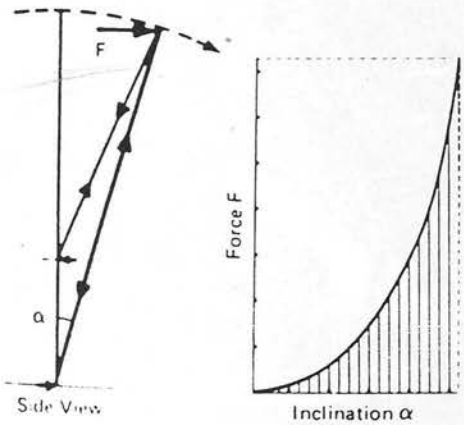


A
Side view of building at night (Photo: Otto Baitz)

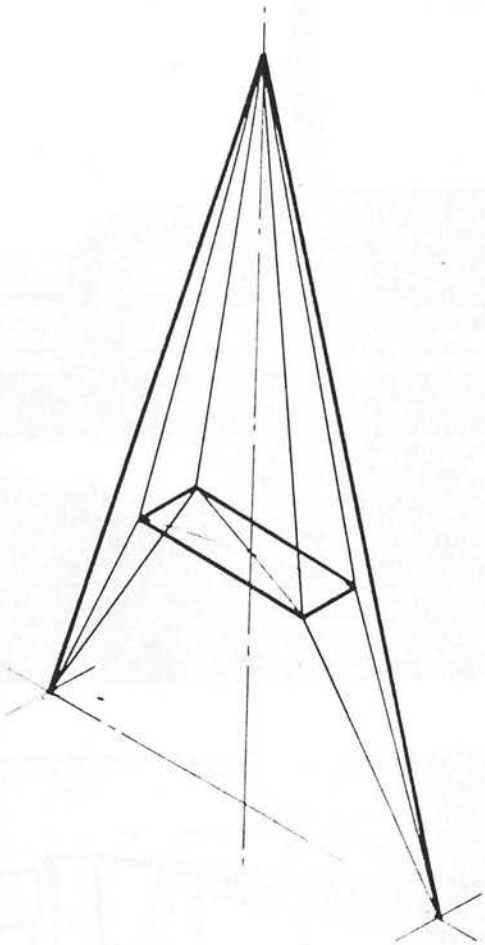


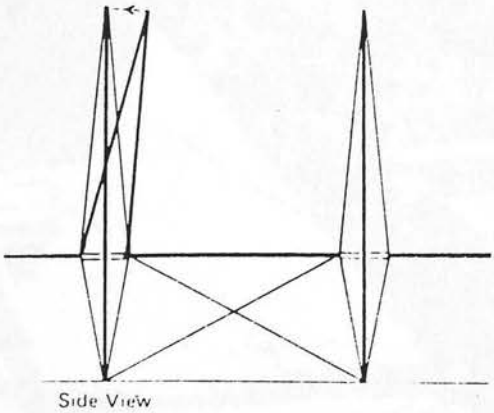
B



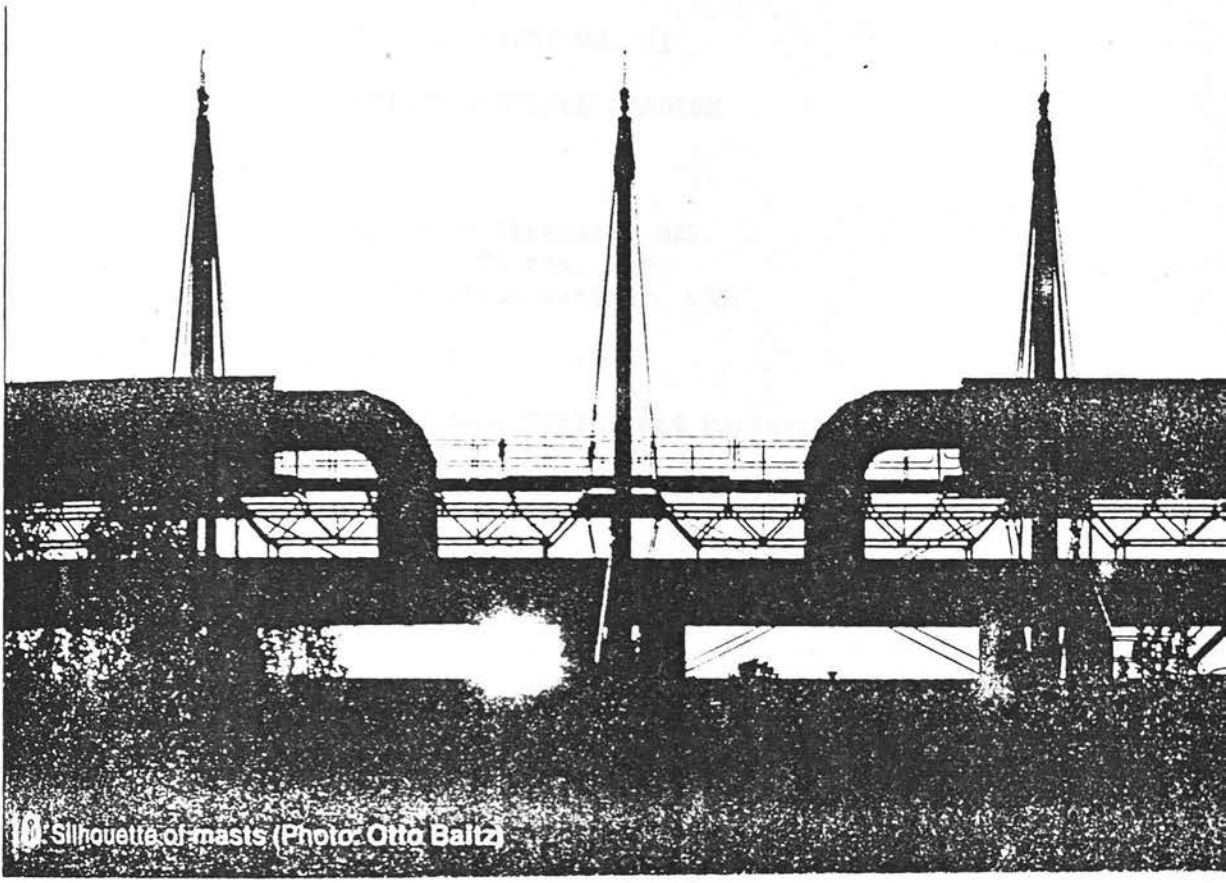


9a Platform support hangers in the same plane as the mast

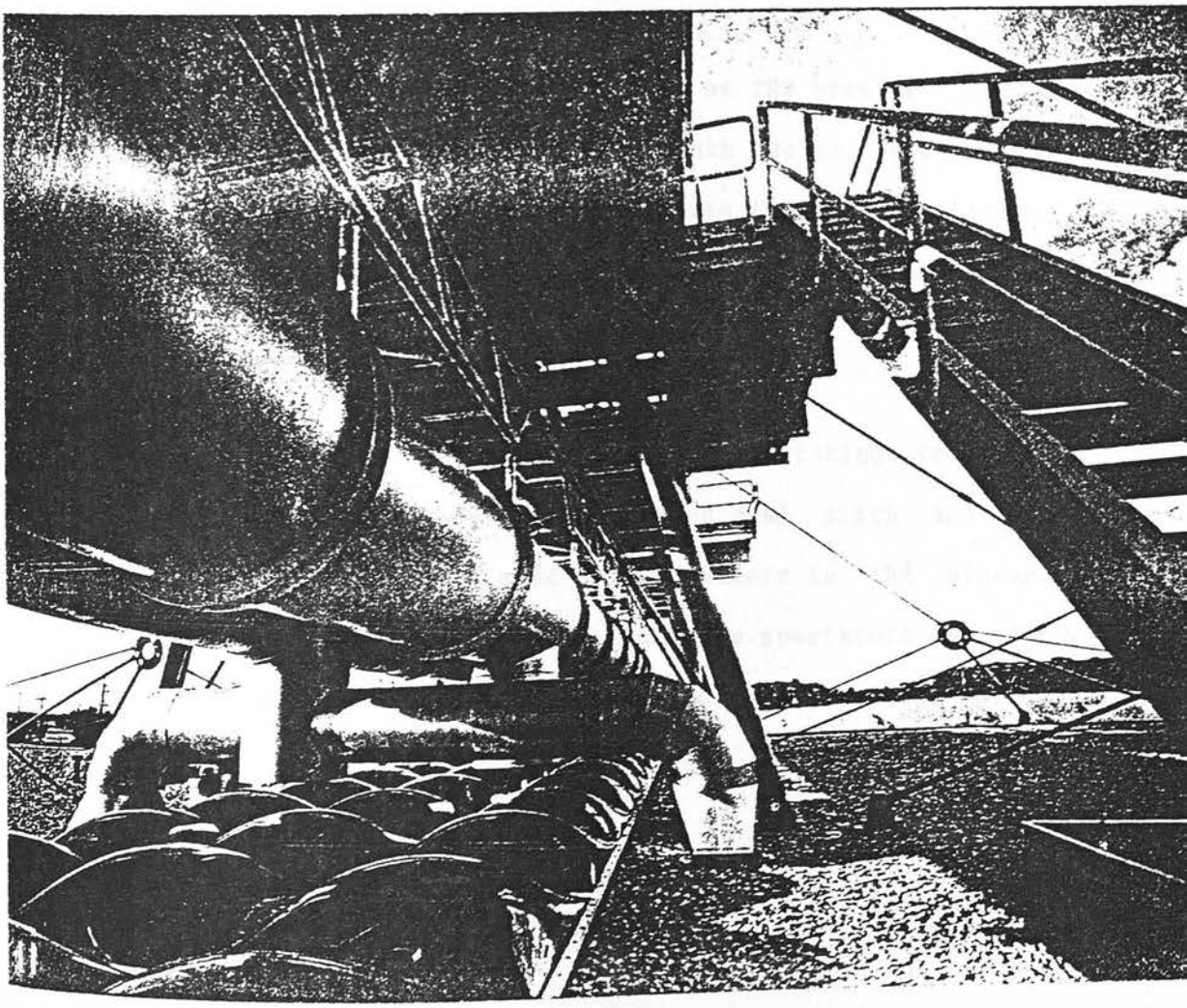




9b Final mast and platform support geometry



10 Silhouette of masts (Photo: Otto Baitz)



CASE STUDY NO. 21

SYDNEY FOOTBALL STADIUM

Extracts from a report by Peter Thompson, OAP
Bill Thomas, OAP
Tristram Carfrae, OAP

Ref: Arup Journal, spring 1989

Architect: Philip Cox, Richardson Taylor and Partners

Engineer: Ove Arup & Partners, structural engineer

1.0 Background

In 1985 the Sydney Cricket and Sports Ground Trust decided to develop new football stadium with a capacity for 40 000 people, to be completed for the Bicentennial celebrations in January 1988.

The Trust envisaged the Stadium not only as the premier playing venue for rugby and soccer in New South Wales, but as an integrated sports compled comprising sports training facilities, tennis and squash courts, swimming pool and car-parking, all contained within a formal garden setting.

A cricket ground is not in fact ideal for watching football, apart from Australian Rules which uses an oval pitch and was developed as a game to be played by cricketeers in the winter. Games requiring a rectangular pitch leave the spectators too far from the combatants when they are played in the middle of a criket oval.

2.0 Utilitarian influences on form

2.0 Utilitarian influences on form

The client brief not only required all 40 000 people to be seated, but also that 25 000 of them should be under cover. It was decided by the design team at the outset that the seating would be of an 'all round' or bowl form and that the roof would be in a continuous strip around the stadium. While this is the only stadium of this form in Australia, notable examples exist around the world such as Wembley Stadium, Santiago Bernabeau in Madrid and the Olympic Stadium in Seoul.

The preferred location for watching football of all types is opposite the halfway line and the more spectators that can be massed about this location, the better. It is also preferable that everyone should have good 'sight-lines' - that is they should be situated less than 90 m from the centre sport, have a clear view over the people in front, and face towards the middle of the pitch.

This was achieved by using tiers of seating which were slightly curved on plan and in section. These tiers were arranged in a continuous terrace around the pitch at the lower level, with two crescent-shaped grandstands on either side of the pitch at a higher level, the whole extending back to a circular boundary. (Fig 2).

This gave rise to an undulating perimeter from which a continuous 'saddle form' roof was generated, rising high above the grandstands on the east and west sides of the pitch and curving down to the two ends. A cantilever of 30 m at the halfway line,

reducing to 10 m behind the goalposts, was required to cover the specified number of seats without visual interruption. The resulting roof form gave the stadium its distinct and exciting character and provided the major engineering challenge.

The chosen form had additional advantages. First, it reduced the mass of the building at the north and south ends in sympathy with the residential area on Moore Park road and the Cricket Ground and secondly it allowed floodlighting to be positioned along the front edge of the roof, so that light spillage - a constant source of local irritation at the Cricket Ground - would be avoided.

3.0 Seating

A pedestrian concourse runs around the stadium at entrance level. A continuous band of open terrace seating runs betwen the concourse and the pitch, which is depressed about 5 m below grade. This seating is constructed from in-situ reinforced concrete, 150 mm thick laid directly onto compacted subsoil and fill material.

Above the concourse on the east and west sides are the crescent-shaped grandstands, which cantilever 10 m out over the concourse and lower terracing. Individual seats are supported by L-shaped tread and riser units of precase concrete spanning 8.5 m between fabricated steel girders which rake down towards the pitch.

Because of the varying geometry of these raking girders, economic considerations favoured the use of structural steel. In any case

the contractor felt that the non-standard formwork required for concrete raking beams simply could not be constructed within the limited time available.

The precast units are detailed in such a way that a waterproof lap joint is formed with the unit below, while the gap between units at the same level is sealed with a silicone sealant. A drip gutter is provided on the upper surface of the raking steel girders to act as a back-up system should the sealant fail or be damaged by vandals. Thus a waterproof construction is obtained without the need for any applied coatings.

The steel girders are in turn supported by a conventional concrete framework which houses all the facilities and amenities which make the Football Stadium an integrated sports facility (Fig. 6),.

4.0 Development of roof design

4.1 Design criteria (Pre-tender)

During the tender period, three structural options were examined in as much detail as time would allow and assessed against the following criteria:

1) Cost
2) Ease and speed of construction
3) Dramatic form and aesthetic quality
4) Environmental impact

The roof types examined were:

a) Simple cantilever using beams or trusses

b) Rafter beams supported by a tensile suspension system

c) A three-dimensional system using the form of the roof surface, perhaps a cable net.

Due to uncertainties of construction and the associated risk to the contractor, who had a fixed date of completion, option(c) was soon discarded.

After further (but necessarily brief) evaluation, option (b), the suspended beam system, was chosen because it certainly gave a more dramatic form and, after a cost benefit analysis, appeared to be more economical than option (a), the simple cantilever.

The initial design used 760 UB rafters (the largest size regularly rolled in Australia) at 8.5 m centres around the periphery, supported by single tension rod stays which passed over tubular masts and outrigger elements to form vertical tie-downs.

It was soon found that there were benefits in passing the tie-downs over another set of outriggers at a lower level, and anchoring them back into the concrete structure below the main grandstands. This eliminated the need for large tension foundations and also produced a stiffer suspension structure by removing the large extensions generated in the long, high-stressed tie-downs.

These rafters supported purlins which were clad on top with lightweights profiled metal decking and would possibly support a

ceiling. Whilst it was recognised that the uplift caused by wind action would be greater than the self-weight of the roof, it was planned to provide counter-weighting where necessary (Fig. 7)

4.2 Roof design: Post-tender

Following the acceptance by the Trust of Civil & Civic's design-and-construct proposal, detailed design was carried out on the roof. The main aims were to reduce member sizes to a minimum and to eliminate the counter weights if possible.

Up to this time member sizes reflected the engineers conservative estimate of wind-loading based on previous experience of conventional cantilever roofed stadia and grandstands. Given the unconventional form of the roof it was essential that wind tunnel testing be carried out. A 1:200 scale aeroelastic model was therefore built and tested at Monash University in Melbourne, using member stiffnesses deduced from the three-dimensional computer modelling.

A slot was introduced into the roof, just behind the leading edge, to reduce the peak pressures in this location by 'bleeding' the wind separation bubble. This technique was used successfully in the design of a grandstand roof at Parramatta two years before where an overall reduction in wind loads of 25% was observed. This slot allowed the floodlights to be mounted behind the roof fascia to give a clean edge to the front of the roof, and when decked with an open mesh also served as the maintenance walkway.

The elimination of counter-weighting required that the suspension

system be conpetent to resist the compressive forces which occur under uplift without buckling. The simple system on which the tender was based was unsuitable, as in order for the stays not to buckle, their diameters would have to increase to a point where they became visually unacceptable.

To overcome this, adjacent suspension systems were inclined together, their front stays were bifurcated to provide two support points for each rafter, and secondary bracing was provided at the point of bifurcation. This arrangement reduced the individual effective lengths and allow small-diameter, thick walled tubes to be used which could accept the compression forces and yet allow the total system to remain visually slender and elegent (Fig. 8).

The further benefits were important by-products of this change. Firstly, the linking together of adjacent pairs of suspension systems to form stable A-frames greatly simplified erection. Secondly the rafter beams, being supported at two points rather than one, reduced in size from a 760 UB to a 610 UB, saving some 300 tonnes of steel.

This reduction in roof beam size altered the fundamental mode of vibration of the roof from one similar to a simple cantilever involving the whole suspension system, to one involving only deformations of the rafter beams with a very different deflected shape. This is turn undermined the principle of using a single quasi-static wind load. Instead static and dynamic effects had to be separated which made the prediction of structural

behaviour, a more complex matter.

In order to ease construction and minimize problems due to fabrication tolerances and temperature effects in service, the roof in its final form is made up of statically determinate, independent structural systems at alternate bays, with infill bays consisting only of purlins (Fig 12). In fact the whole roof was detailed with no special facilities for dimensional adjustment (apart from an ability to align the leading edge of the roof which was not used) and was erected without problems.

In the end, it was decided to use a perforated aluminium cladding to create a smooth soffit to the roof. The perforations act both to increase acoustic absorption and allow the wind pressures to pass through and load the structural cladding above. This ceiling improved the roof's appearance and prevented pigeons from roosting on the rafters. Ingenious detailing of the purlin system allowed installation of both layers of cladding from above without the use of scaffolding. In fact no scaffolding was used for the entire roof erection.

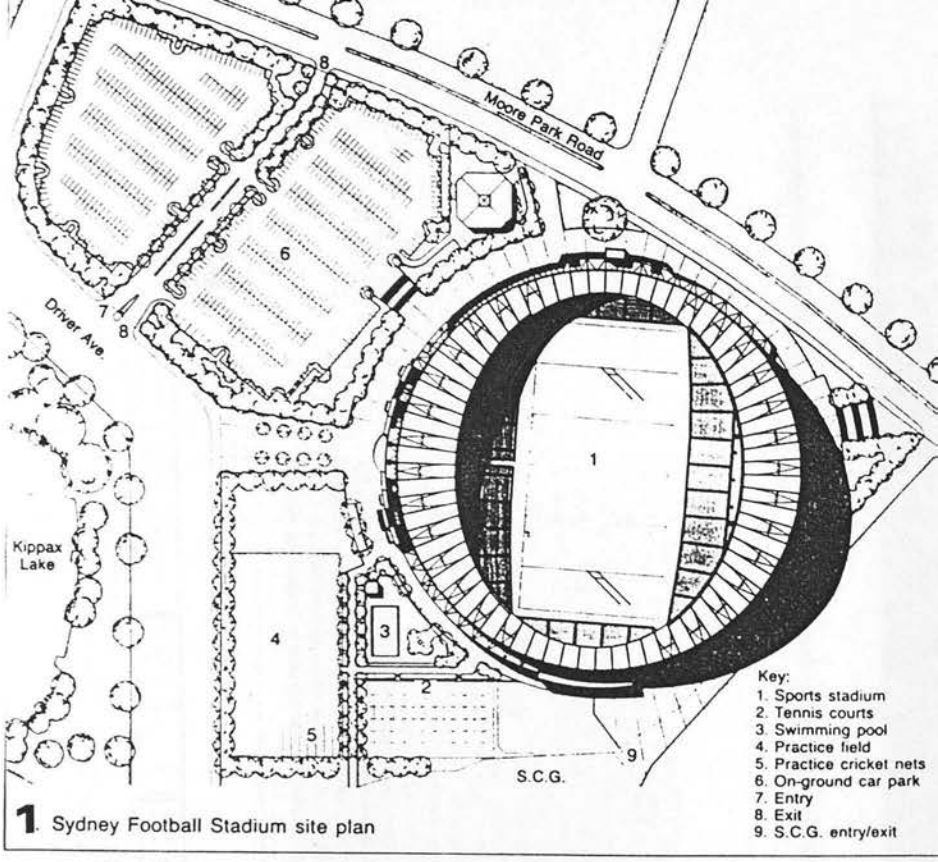
The structural system for the roof is completed by space trusses located below the grandstand seating which deal with the forces caused by the mis-alighment of the radial roof beams with the grandstand beams (which follow the more gentle curve of the seating arrangement).

The elegant form of the roof is reflected in the connection design. The typology of the roof support structure varies around the roof perimeter, from the triangulated masts and tie-downs

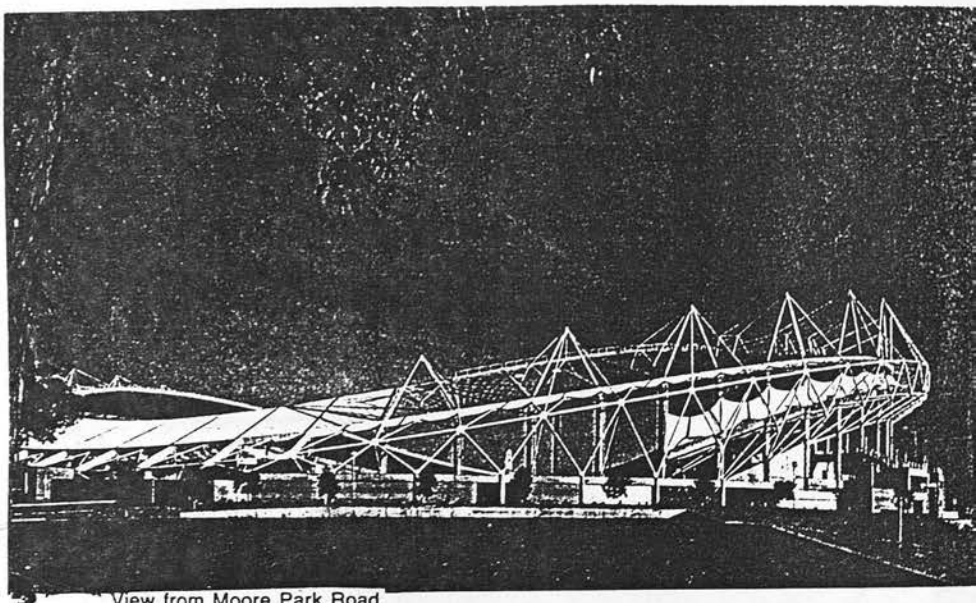
along the east and west sides to simple cantilevers with vertical tie-downs at the ends where the span is considerably less. The detailing is such that these transitions occur smoothly and enhance the overall 'swoop' of the roof.

Each connection after structural sizing was modelled by the architect with scale models and three-dimensional computer graphics. The joints were then varied where necessary to achieve a satisfactory architectural and engineering result.

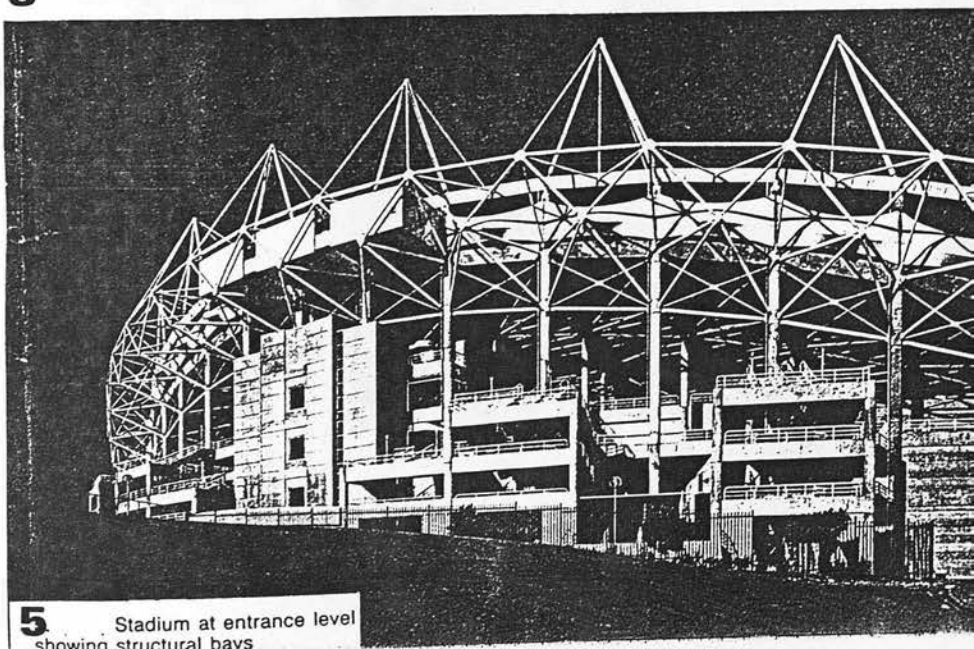
To add a visual contrast to the elevation and minimize wind intrusion, the gap between the upper edge of the grandstands and the rear edge of the roof is partially covered by tensioned fabric structures. The membrane is a pvc-coated polyester fabric which is tensioned by galvanized steel cables attached to the roof support structure.



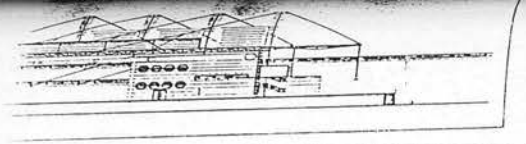
1 Sydney Football Stadium site plan



3 View from Moore Park Road



5 Stadium at entrance level showing structural bays



Ground and the Rotunda by cars, crowds and arc lights, the area already harassed by cars, crowds and arc lights, the proposal to build was met with considerable resistance. Cox's response was to design a swirling fun-fair building. The cantilevered steel roof structure is spectacular, supported on a tubular steel triangulated space-frame truss. Concerned with scale, Cox pulled the northern rim of the roof down in salute to the residences and the major traffic route that lie to that side. The corresponding southern point was also dipped in a token gesture to the weather conditions that threaten from that

14

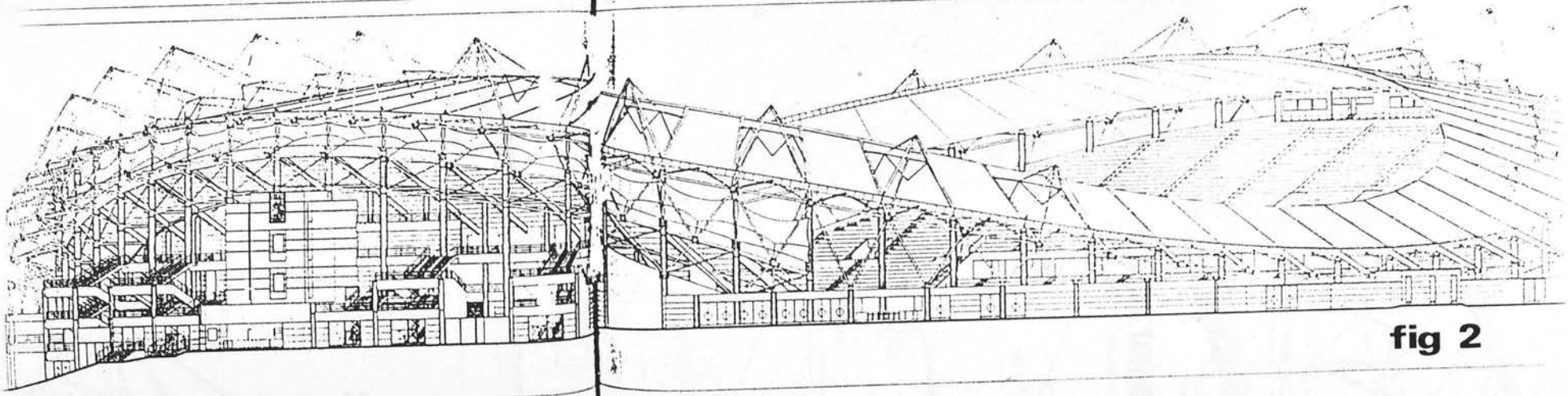


fig 2

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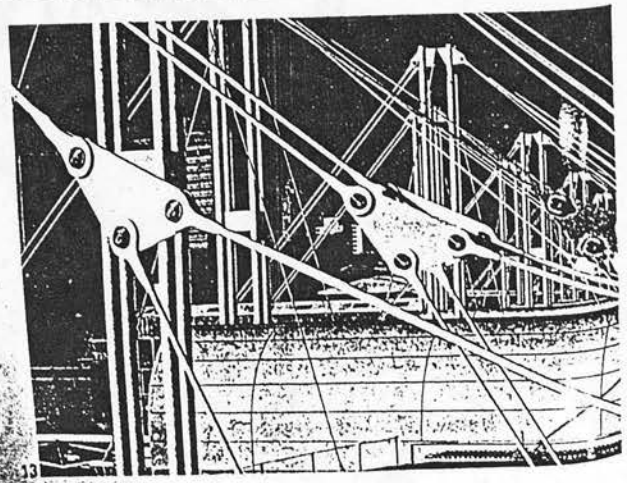
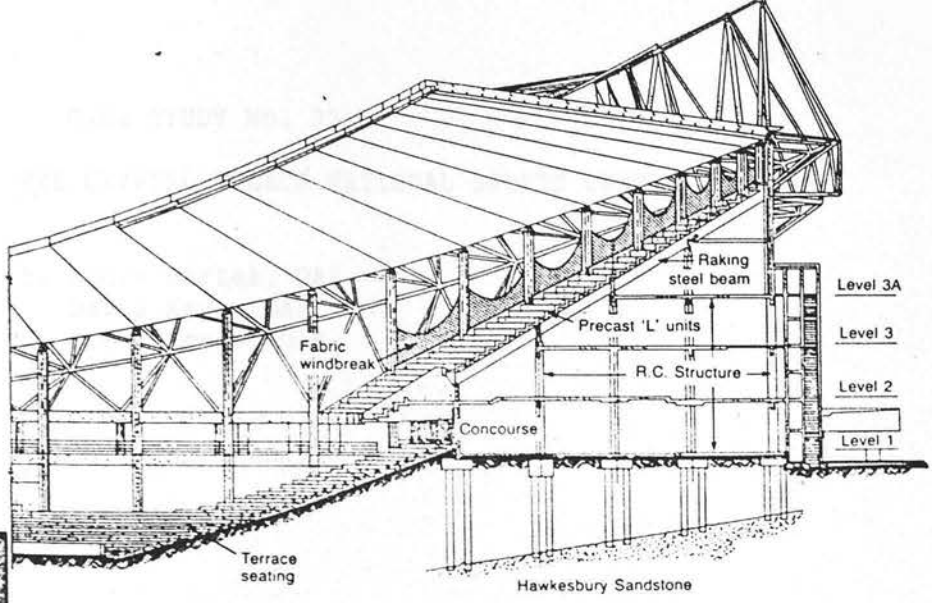


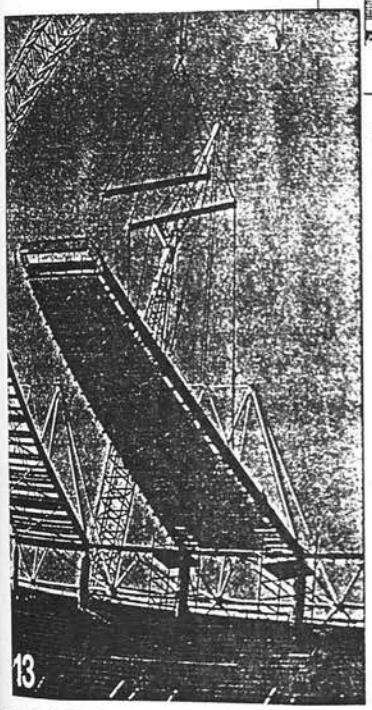
13

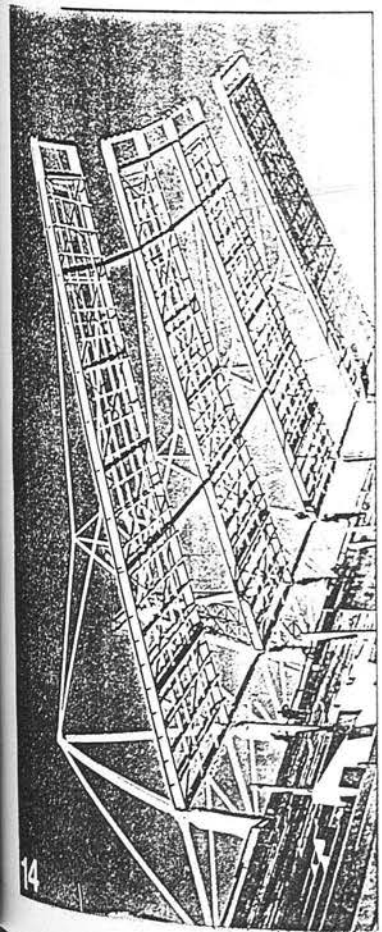
direction.[5] The rectangular playing field runs north-south so the roof swells up and out to the east and west over the grandstands located where viewing is optimal. Wind-breaking bands of taut folded fabric wrap like great scarves around high sections between the roof and the walls, adding to the mad celebratory air of the building.

As the roof sweeps up and dips down it defines a space unified and continuous and never static. As in the National Athletics Stadium, the brick-faced concrete seating area (for 38 500 spectators) is solid and heavy, creating a great scooped-out base for the wild white steel bonnet perched on top. The building peers out from under its bonnet with the somewhat stunned expression of a dowager bewildered by her own audacity. More poetically, Cox sees the roof as a parasol or a white cloud floating above the earthiness of the base. Despite little connection between the roof and the walls, the forms create a powerful sense of enclosure. The playing field is submerged three metres below the natural ground level

13-14.
structural bays in the course of erection

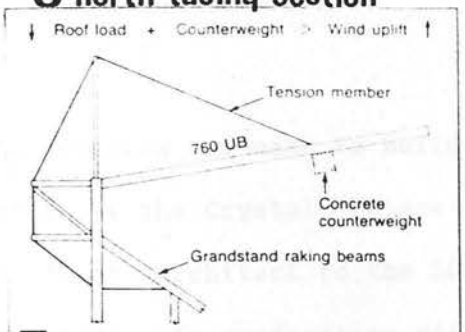




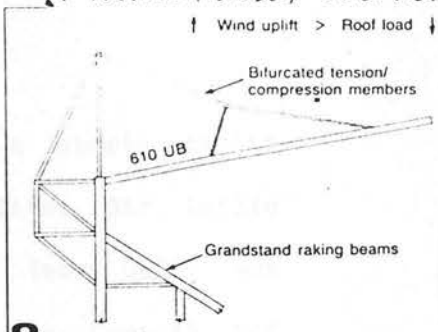


6 north facing section

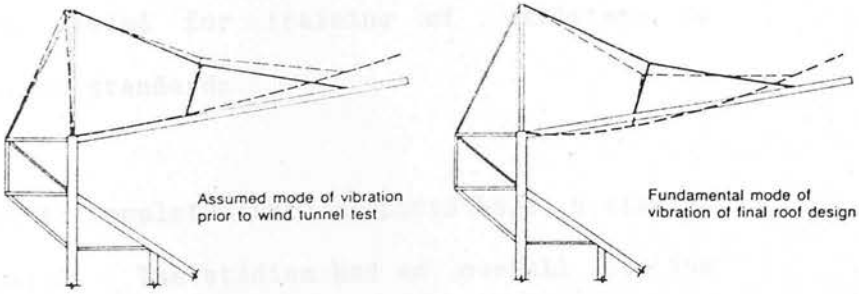
c. North ramp section



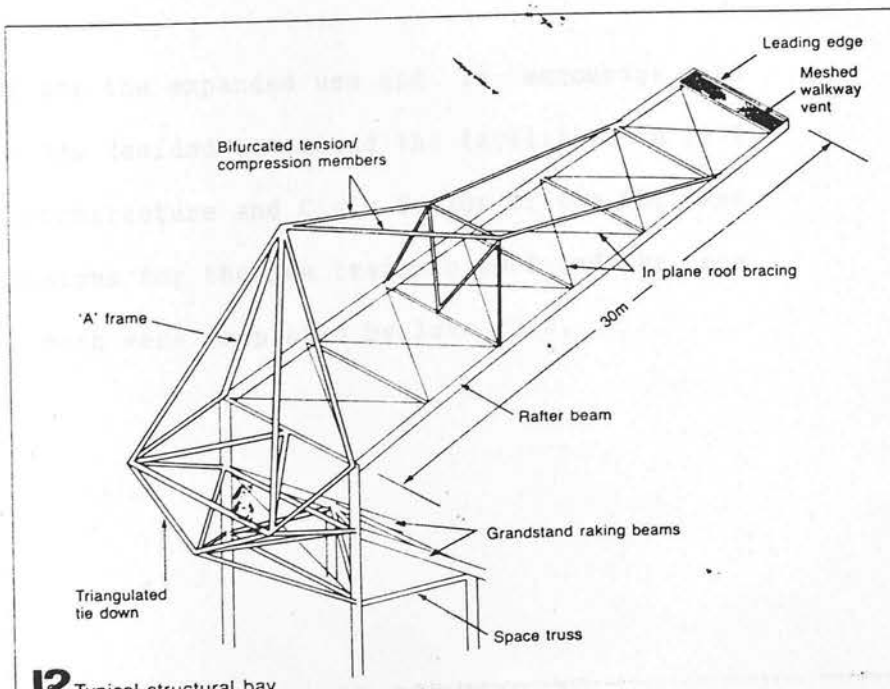
7 Pre-tender tension structure



8 Post-tender tension/compression structure



11 Modes of vibration



12 Typical structural bay

CASE STUDY No. 22

THE NEW GRANDSTAND AT THE CRYSTAL PALACE NATIONAL SPORTS CENTRE

Extracts from a report by Andre Bartak, OAP
David Kaye, OAP
Time George, OAP

Architects: LCC

Structural Engineers: OAP

1.0 Introduction

1.1 Background

When in 1953 the decision was made to build a sports centre worthy of the nation at the Crystal Palace Site, Sir Leslie Martin, then the Chief Architect to the LCC (now GLC), was commissioned to prepare, in conjunction with the Council for Physical Recreation, a scheme for a centre capable of providing all the facilities needed for training of athletes to international and Olympic standards.

In 1964 the main elements completed were a sports hall, a stadium and an athletes' hostel. The stadium had an overall seating capacity of 12,400 of which 4,000 seats were under cover.

1.2 In order to cater for the expanded use and to encourage its future increase, it was decided to extend the facilities in 1972 The Department of Architecture and Civic Design of the GLC was briefed to develop designs for the new training pool and the new covered grandstand. Both were completed by late 1976.

2.0 Description

The new grandstand is set out on a large radius curve, is 122 m long and 27.4 m wide. (Fig 1) It consists of a concrete superstructure founded on piles, and covered by a structural steel canopy (Figs 2 and 3).

The terracing is of non-structural precast concrete seating units supported by a 254 mm thick in situ sloping suspended slab spanning between main reinforced concrete frames mediate floors are of ribbed slab construction. The roof of the canopy incorporates 16 welded plate girders, each 22.1 m long overall and spaced at approximately 7.6 m centres. These girders are primarily cantilevers, vary in depth from 1.07 m to 0.38 m at the tip, Universal beams spaced at 3.05 m centres span between them and carry the roof decking. The stressed skin PVC coated metal trough decking is connected to the purlins by means of special watertight fasteners.

Eight large pyramid-shaped assemblies of steel tubes, springing from the back terrace behind the seating, offer support to the roof from above, via the inclined front ties. At their ends they are of 219 mm diameter, but the long central portion of each member is of 245 mm diameter. The transition pieces are in the form of reducer cones rolled up from plates.

The principal support element of each assembly is in the form of an A-frame. The legs of these frames are of welded box sextion. Each box is stiffened by internal plates. At right angles to the plane of the frame the legs have a tapered appearance.

The back ties of each pyramid consist of two round tubes, 219 mm diameter, 9.5 mm thick. These ties are anchored to the top of the concrete construction and the base details consist of a steel collar of two plates separated by six stiffeners (fig 4)

The capping piece of each pyramid is a complex box, shaped to form the apex of each A-frame (Figs. 5 and 6).

3.0 Design considerations

3.1 General

The new grandstand was to be situated on the opposite side of the track vis a vis the existing covered seating. It was to contain 5,000 seats, thus more than doubling the number of covered seats in the stadium. The brief required the new structure to resemble the existing one as closely as possible, which was simple and effective in that it exploited the natural site topography as a basis for its structural form. The new siting did not provide the designers with a similar opportunity and a reinforced concrete superstructure was necessary in order to provide the terraced seating required. This meant, as can be seen in Fig. 8, that the back ties are anchored to the top of the concrete superstructure, rather than to a concrete counterweight in the ground, as was the case for the existing stand.

Geometrical reasons for the new stand necessitated the use of two back ties per support assembly as opposed to the single back tie employed before. The A-frame compression members, previously constructed in concrete, were changed to steel box sections.

This was considered appropriate, particularly from the point of view of erection.

These comparatively small departures, however, still enabled the basic geometry of the roof to remain. The retention of the visual leitmotiv provides a link between the existing and the new stands and reflects the consistency and the continuity of the architectural design (Figs. 8 and 9).

3.2 Lateral stability

The architect required that there should be no diagonal bracing in the plane of the roof. In order to provide stability against side loading of the cantilevered portion of the roof, the canopy was initially designed as a frame on plan, with HSFG bolts used to provide moment-resisting connections at the purlin/girder junctions. there was little in-plane stiffness inherent in this arrangement, and relatively large deformations could be expected. Up until then the stiffening effect of the roof sheeting had been ignored, though clearly if the deck to structure fixings, were adequate, a substantial contribution would be made by the sheeting.

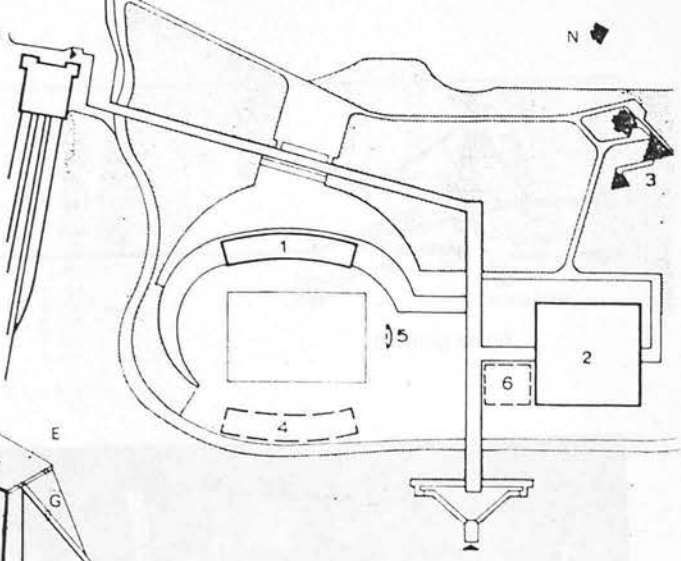
in recent years theoretical and experimental work has been carried out on the employment of light corrugated metal sheeting in stressed-skin construction. On this basis the design was modified to allow the metal deck to be effective as a shear diaphram, and the maximum horizontal displacement of the outermost purlin was reduced to about 7 mm. One consequence of the method is that the connections between the sheeting and the

supporting steel occur at the valleys of the troughing. To ensure watertight connections the fasteners were provided with special sealing grommets.

holding down detail

Key
1. Existing stand
2. Sports hall
3. Hostel
4. New stand
5. New scoreboard
6. New training pool

Fig. 1
Site plan



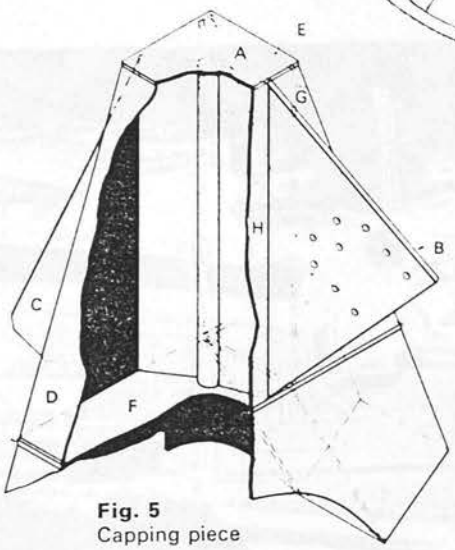
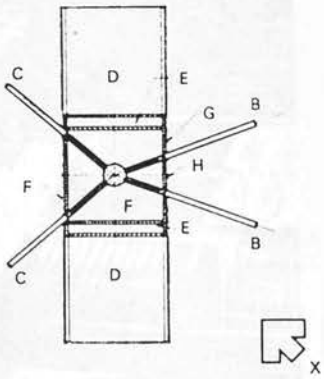
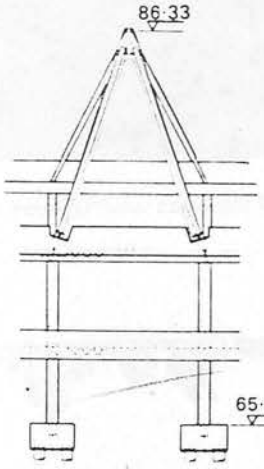


Fig. 5
Capping piece

View in direction X



Plan below plate A



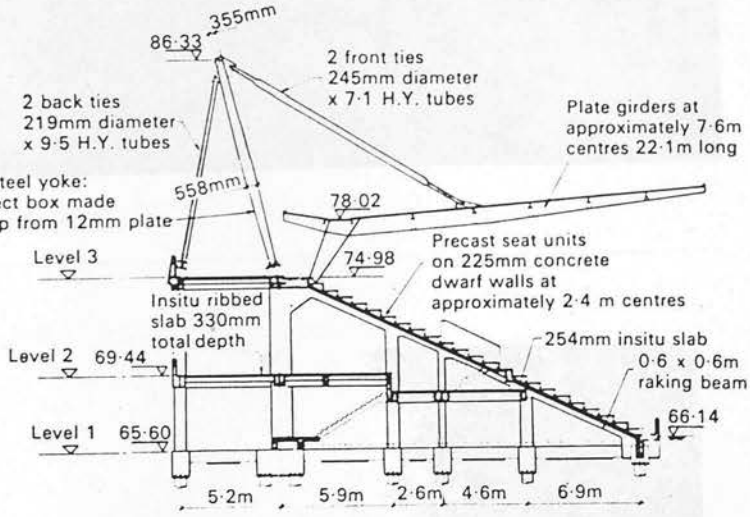
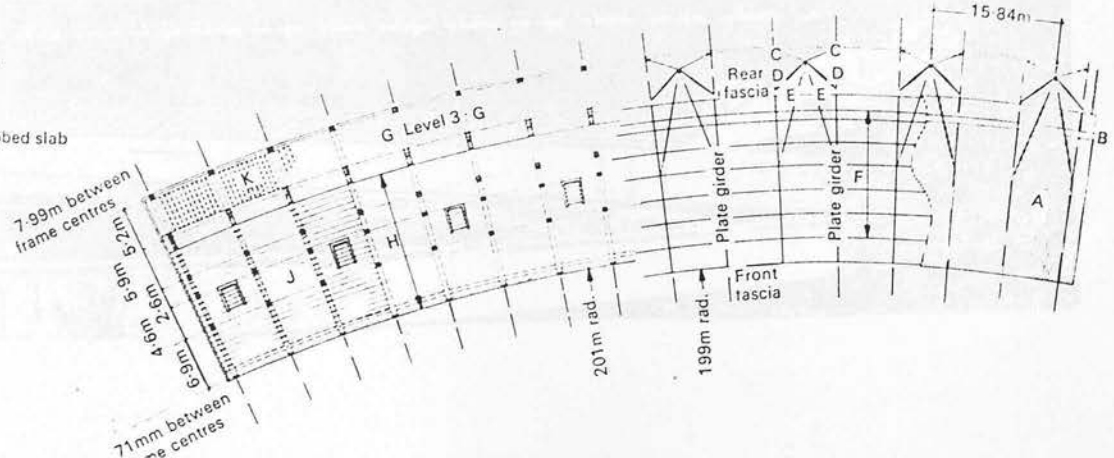
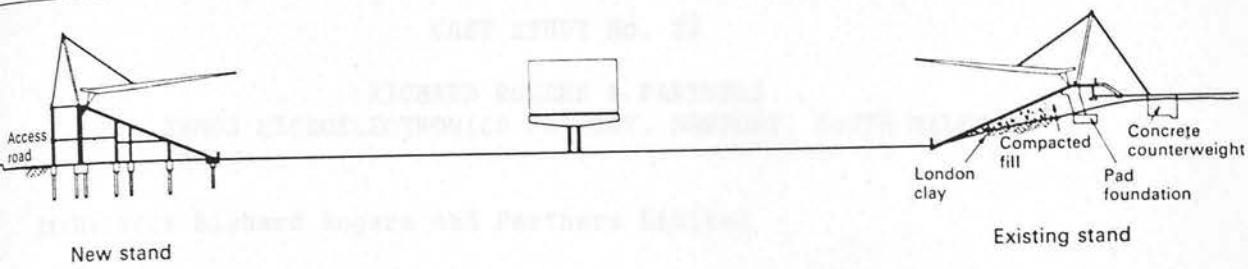


Fig. 2
Cross-section through new grandstand

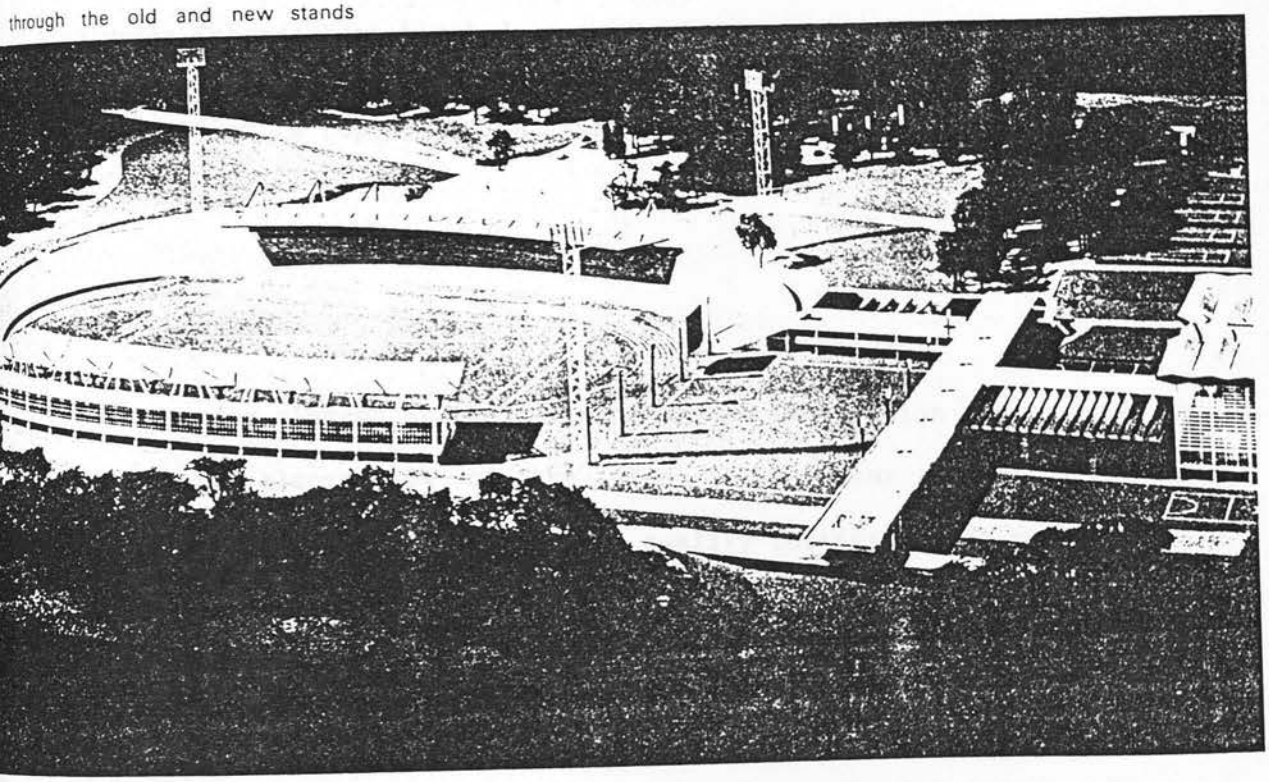
A PVC coated metal decking
B GRP gutter
C Tubular back ties
D Built up box strut
E Tubular front ties
F UB purlins at 3·05m centres
G Insitu concrete frames
H 254mm insitu sloping slab
J Precast concrete seat units
K Insitu reinforced concrete ribbed slab

Fig. 3
Plan

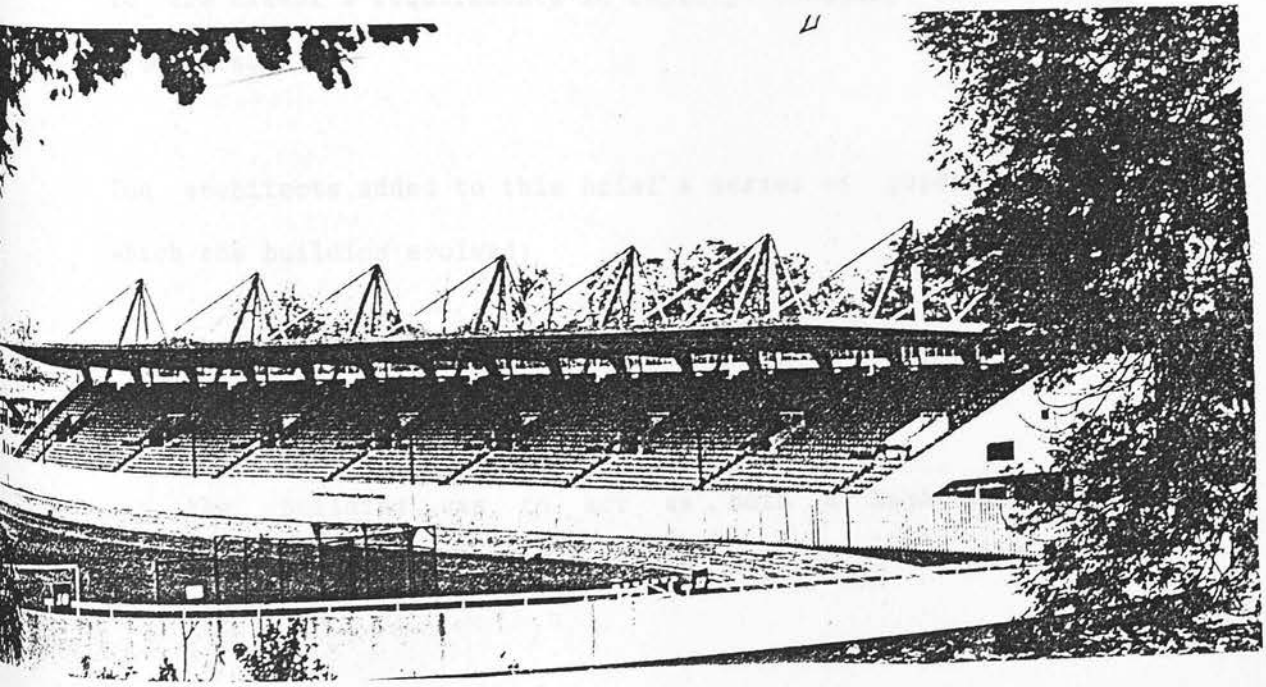




through the old and new stands



h of the architect's model (*Photo: copyright* Greater London Council)



CASE STUDY No. 23

RICHARD ROGERS & PARTNERS
INMOS MICROELECTRONICS FACTORY, NEWPORT, SOUTH WALES

Architect: Richard Rogers and Partners Limited

Consulting Engineers: Anthony Hunt Associates

Steelwork: Tuberworkers Limited

Extracted from a report by Tony Hunt and the A.D. issue on British Architecture

1.0 Background

The major element of the building is the fabrication area which occupied almost 50% of the plan. The other accommodation consisted of offices, restaurant, quality testing, plant room and a maintenance area. (Fig 1)

The nature of the fabrication process called for a controlled interior environment necessitating highly serviced building. In addition, the building required sufficient flexibility to respond to the client's requirements in rapidly changing technological processes.

The architects added to this brief a series of guidelines from which the building evolved:

- the design was to respond successfully to changes in the brief as the programme evolved during construction;
- the building was to act as both a high performance precision, production machine and as a friendly and

stimulating environment for employees;

- the design was to allow for maximum flexibility and for potential growth and change required of a rapidly evolving industry
- the design and construction of the building was to suit the client's fast building programme

2.0 Concept and Development of Structure

The building design evolved as a single-storey steel structure conceived as a kit of rapidly erectable parts with maximum off-site prefabrication. The basic concept of the building comprised a central linear circulation and service spine with lateral wings housing specialised activities. The linear form was based on repetitive structural elements which used to form the present eight 13 x 36 m bays. The structure was designed to extend in the future in linear sequence without disruption to current production which, when fully operative would be on a three shift 24-hour cycle. (Fig 2, 5)

The spine, 7.2 m wide and 106 m long, provided an internal street, with vending machines, public telephones, seating, meeting places, planted areas, library, and information displays, and waiting areas for the offices. The spine provides the visual linkage for security control and is intended to link up with future phases of building on the site, so that all the facilities in the buildings are readily available to staff. Offices and restaurants are on the south side of the spine, and the clean production room to the north.

2.1 The prime constraints determining the structural concept were the provision of column free areas for total internal flexibility together with the capability to accommodate large volumes of services plant. The central spine provided access outside the building envelope and minimised duct distribution runs. The central spine was also developed as a major structural element in consideration of the heavy plant equipment required.

2.2 Steel structure

This approach was developed through various options into the final concept of a multi-level central spine off which a single storey clear span structure was hung from each side forming an umbrella for all activities. This umbrella, in the form of a deep lattice roof structure provided the zone for the distribution of all major ductwork, and supported the suspended roofing system. (Fig 2, 9, 10, 11)

The superstructure was a combination of welded tubular steel lattice elements forming both the 'spine' and the 'wings' of the frame. The structure was designed with pinned connections and cross-braced for stability in both vertical and horizontal planes using single stainless steel shear pins at all connections. The spine consists of 15 m high lattice tubular columns 4.8 m apart at 13.2 m centres, with compression cross braces at the two upper storeys and a lattice portal frame at the lowest storey (Fig 4). The frames are linked longitudinally by I-beams at level 1 and lattice girders at level 2 to form a braced core which supports the wings of the structure and also the plant deck platforms.

Other than the lattice portal frame, general stability was achieved in both vertical and horizontal planes by tension rod bracing with solid body turnbuckles for adjustment.

The wings comprise 38 m long clear span primary lattice trusses. Their inboard end is supported by the spine, the tension hangers give support at one third points and the outboard end is stabilised by a combined tension and compression bipod (Fig 8, 9).

Secondary lattice trusses at 6 m centres span between primaries and support a tertiary grid of RHS beams via hanger which in turn support a profile steel roof deck. Fixing cleats and stud connections are provided at predetermined positions on a regular pattern to provide fixing points for ductwork.

The structural system provides uninterrupted column-free spaces for maximum internal flexibility. The roof is fabricated from 6 m span steel decking with thermal insulation and 5-layer roof membrane. The external walls are based on a system of standardised mullions which will accept most types of infill: ingle glazing, double glazing, transluscent or opaque panels. This gives the client the flexibility of varying walls and finishes. The initial design proposed double glazing on the office areas and solid insulated sandwich panels for the production areas.

3.0 Wind-effects

Possible special wind effects were considered due to the unusual

and exposed nature of the structure, with advice on the determination of wind loading being obtained from the National Maritime Institute. Effects examined in detail included possible wind turbulence across the roof plane caused by the exposed roof trusses and the oscillation mode and vortex shedding associated with the tension hangers. Results showed that no special measures were needed although cost allowance had been made for providing dampers for the hangers until such time as tests could be undertaken in situ to prove the assumptions in respect of the natural frequency of the structure and the applied damping.

4.0 Thermal movement

The structure was designed to accommodate thermal movement by the provision of slotted connections both within the secondary lattice girders and along the spine structure. The connections were located at one side of a bay adjacent to the main girder. Such bays were known as Free Bays. A Rigid Bay was one where no such connection was provided

5.0 Corrosion protection

The protection system for the steelwork consists of a sprayed zinc metal coating followed by two coats of High Build chlorinated rubber paint. The latter selected as overcoating can be undertaken after washing down, without any further preparation, the solvent in the new paint forming a homogeneous mass with the old paint.

6.0 Fabrication

The structure was designed to simplify both fabrication and construction on site. The spine was fabricated as a separate element, brought to site and assembled with simple pin joints and bracing, while all the main roof elements were fabricated ready for attachement to the spine columns and tension hangers.

The detailing philosophy of the single pin connections was to produce a simple-to-erect structure with split pins and washers. The absence of a bolt head and nut together with the use of high strength Firth-Vickers stainless steel material not not makes the pin size appear smaller, but actually reduces the overall size of the connection. In this way, it has been possible to design a cleaner and more elegant joint. (Figs 12 to 16)

Repetition of structural elements has been maximised. As a consequence there are minimal variations to any of the elements thus reducing the number of drawings and jigs required and ensuring correct assembly on site. This approach reduced fabrication time and also increased the speed of erection on site. Strain gauges were attached to the tension hangers supporting the primary girders on three frames, representing the varying structural conditions, ie. end frame, internal and the courtyard frame. This monitoring process enabled a correlation to be undertaken between the predetermined force, camber and catenary and the actual force in the tension hangers.

7.0 Erection

All work on site took place in linear sequence with significant

overlapping of substructure and superstructure, followed by services, cladding, fitting out, etc. The main frame is self-stabilising and therefore following trades were able to commence as soon as the first three bays of the frame were erected. Total time for erecting the superstructure was fourteen weeks, one week less than the programme allowed. (Fig 17)

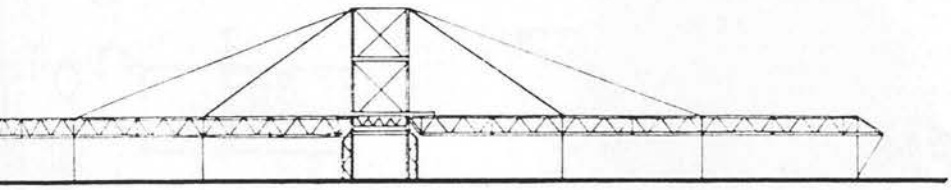
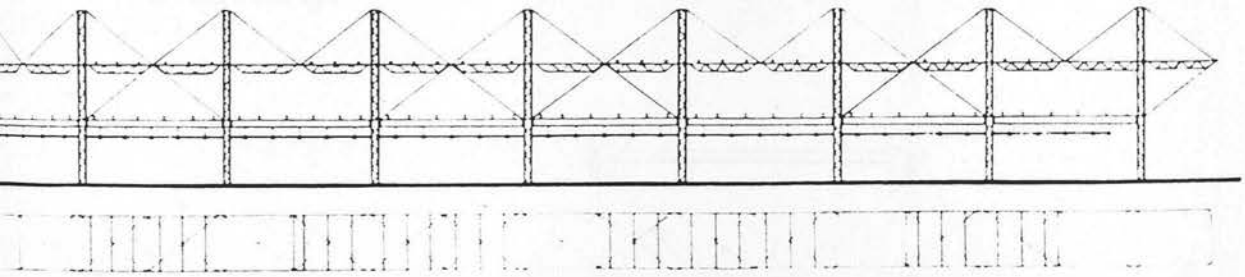


Fig. 1. Coupe transversale
Querschnitt
Cross section

ation de l'épine dorsale et vue en plan du niveau 2 ● Aufriß des „Rückgrats" und
Höhe 2 ● Spine elevation and plan on level 2



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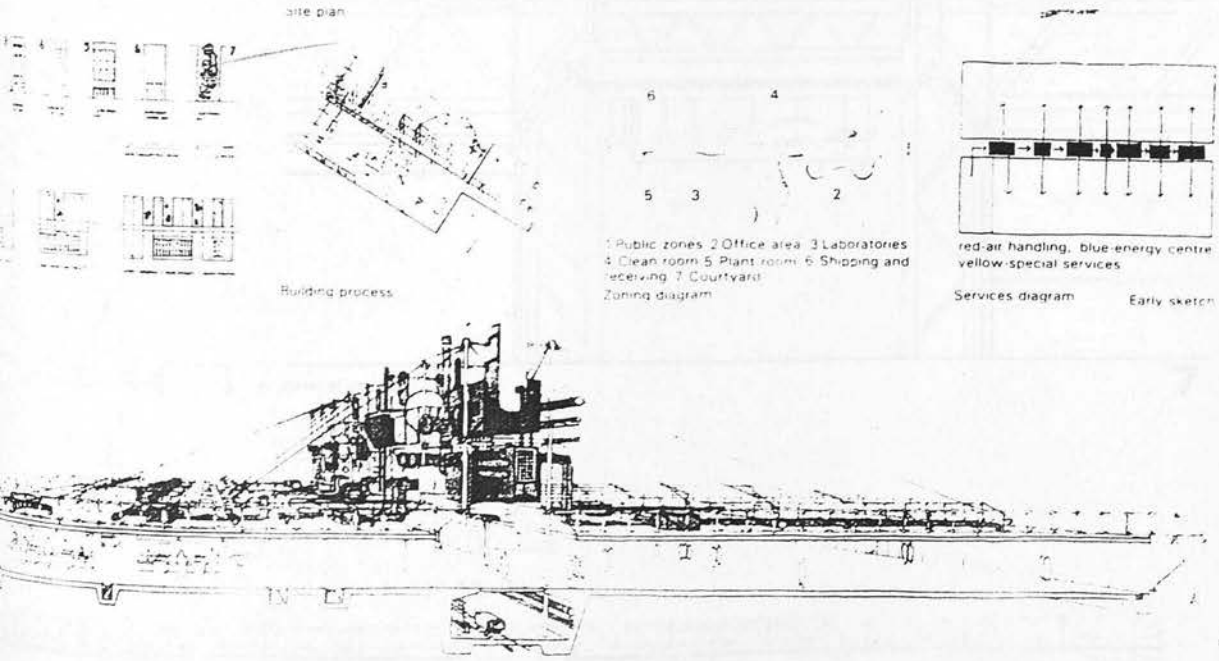

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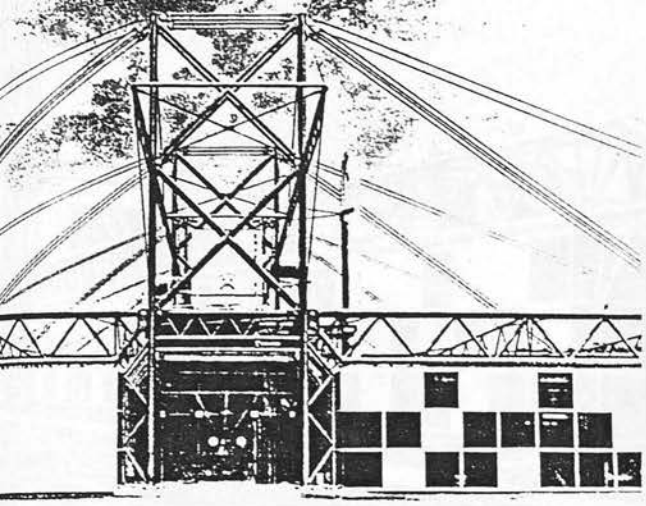


Portiques de « l'épine dorsale » et haubans ● Eingang. Tragwerkrahmen und Zugstabe ● Entrance. Spine frames and tension hangers

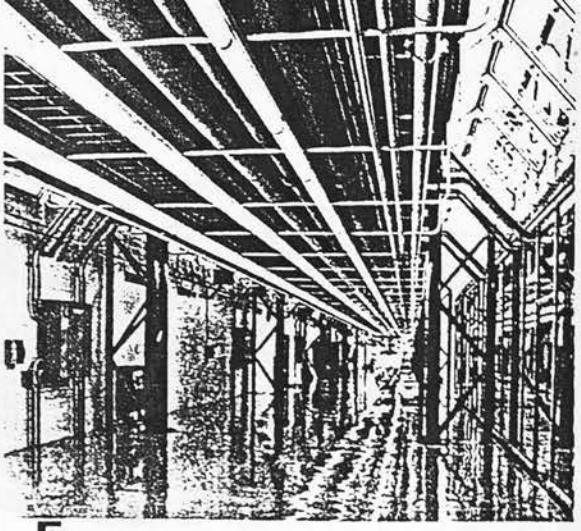
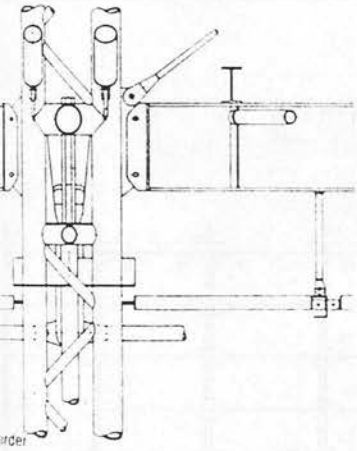
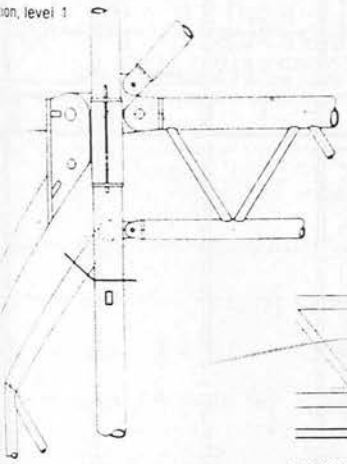
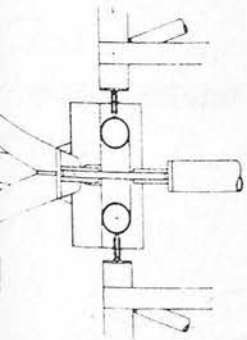
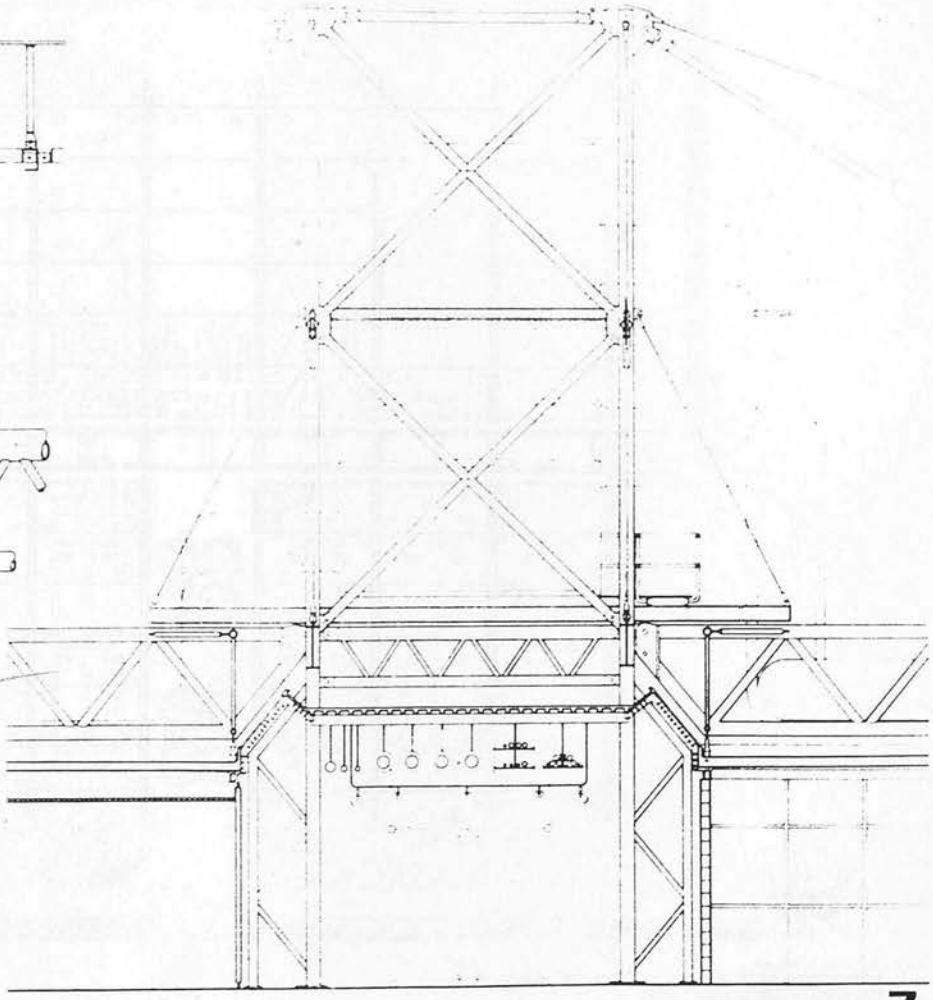


Fig 5 Hall intérieur. Vue en long ● Innenansicht in Längsrichtung ● Internal mall. Long view









2 Detail of elevation

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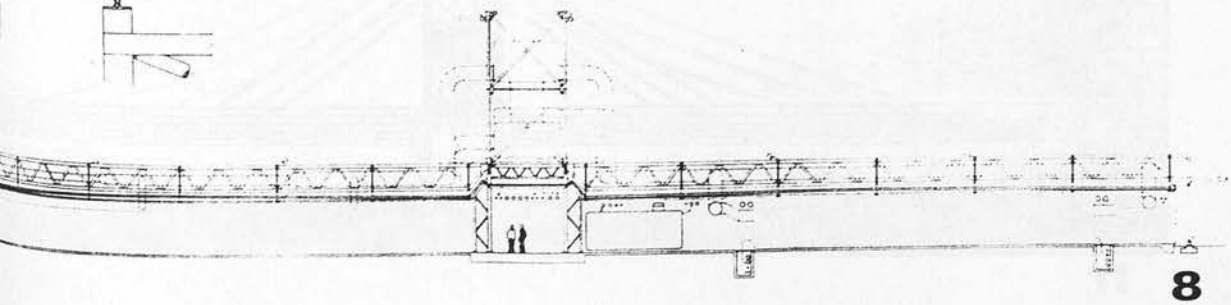

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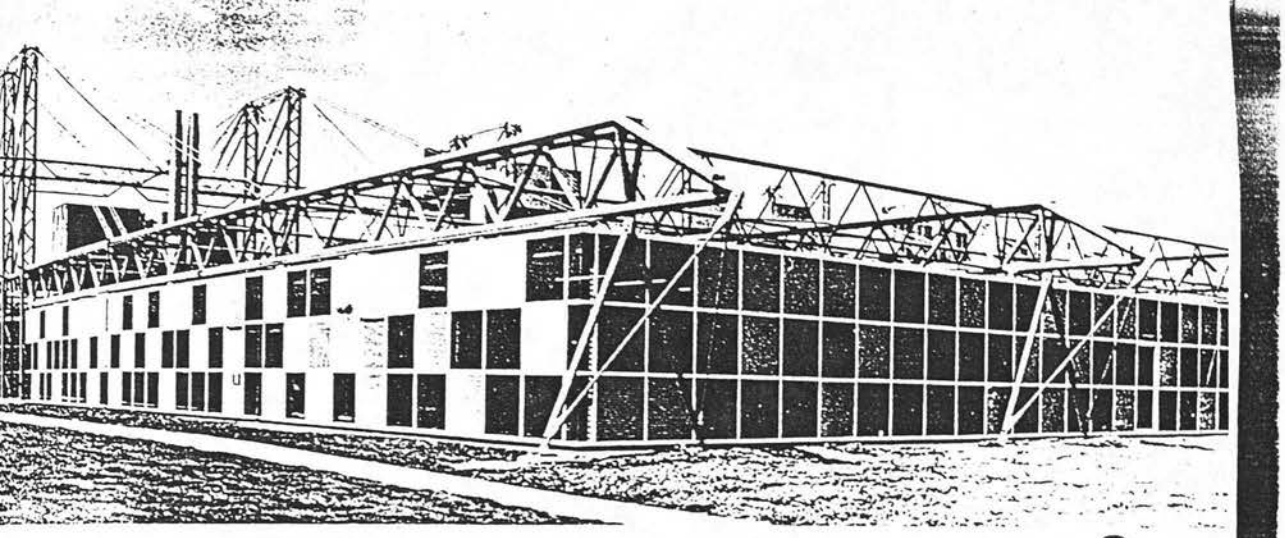
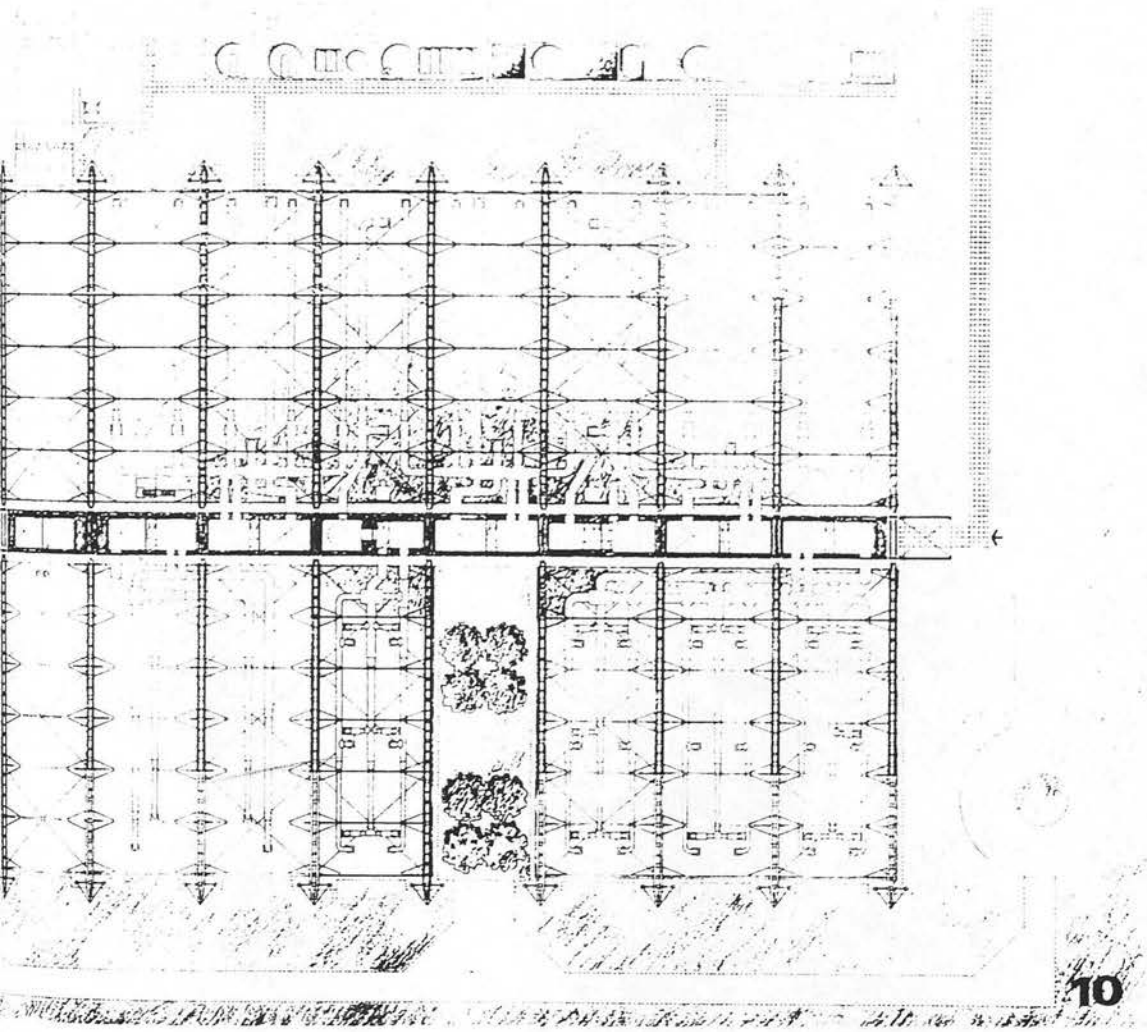


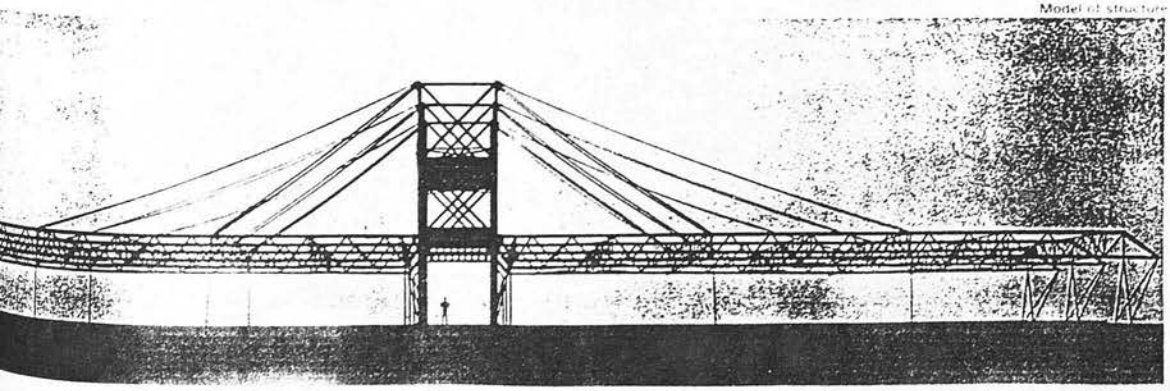
ure Extrémité poutres principales ● Außenansicht. Ende der Haupttrager ● External view. End of main girders

9



10

Modell ist structure



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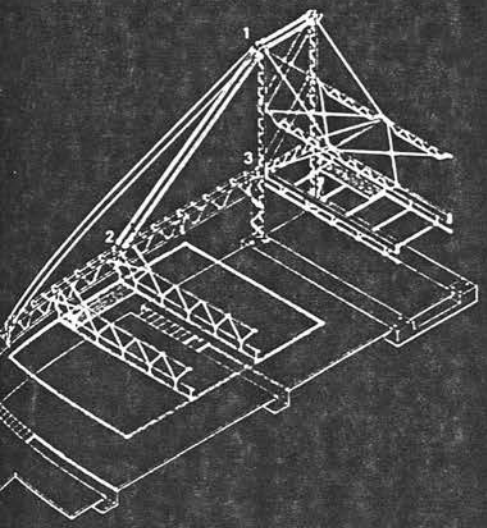
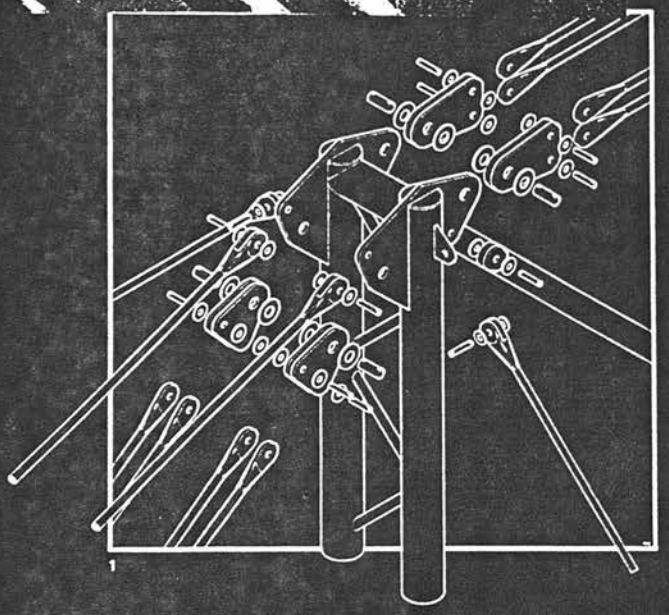


with junction details



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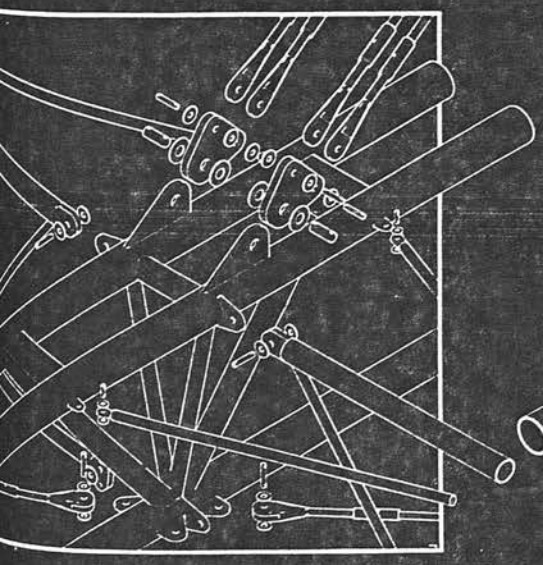

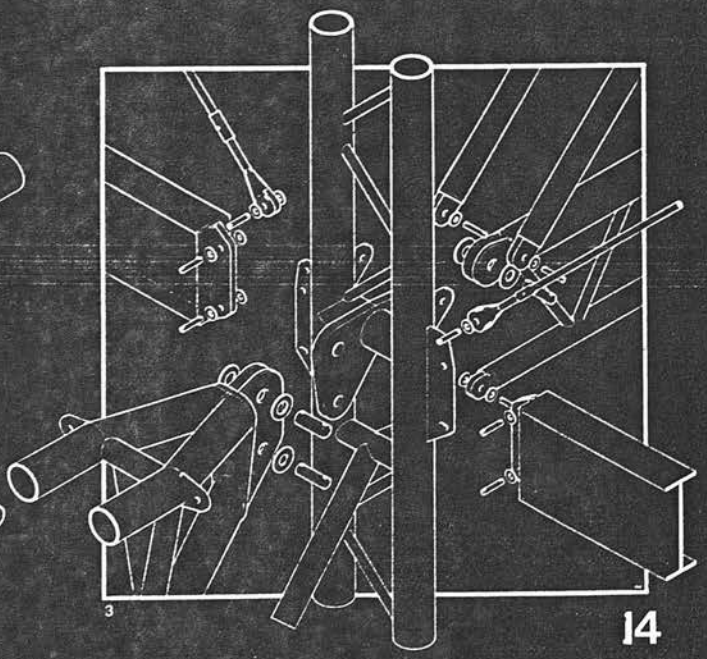

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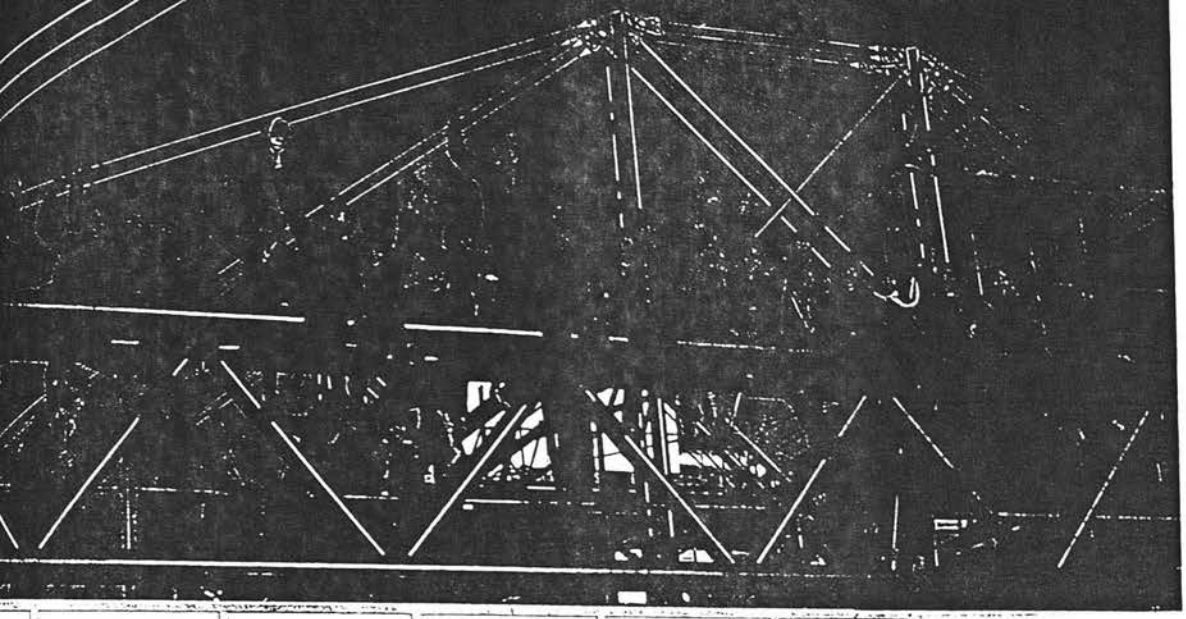
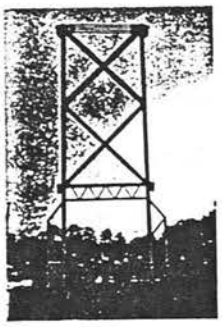
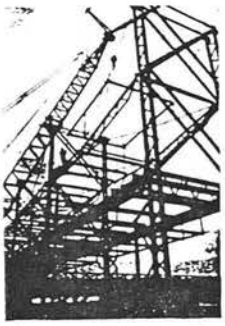
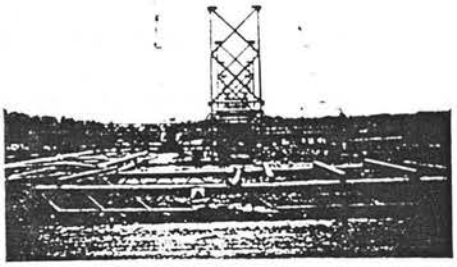
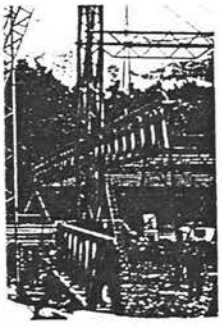


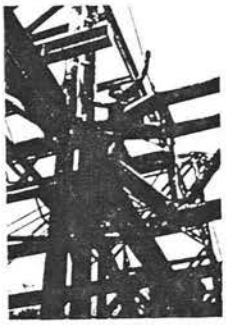


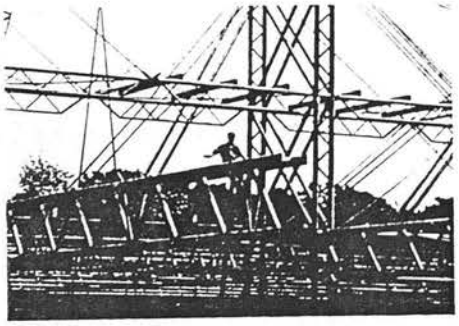


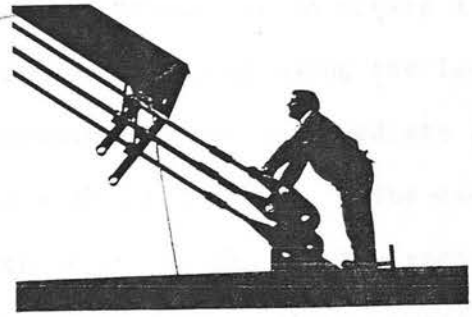


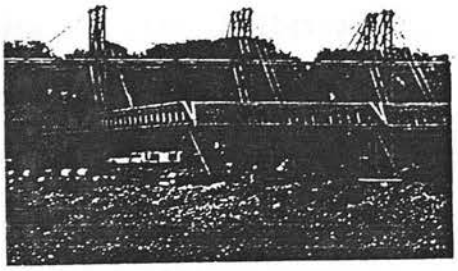
17 CONSTRUCTION SEQUENCE

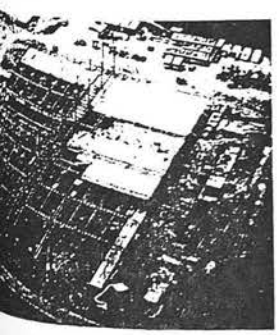


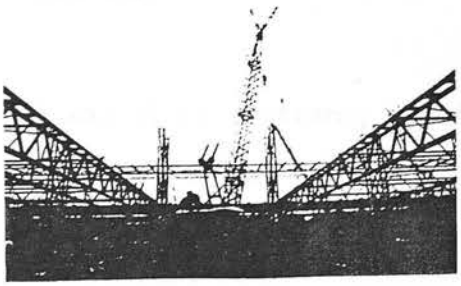


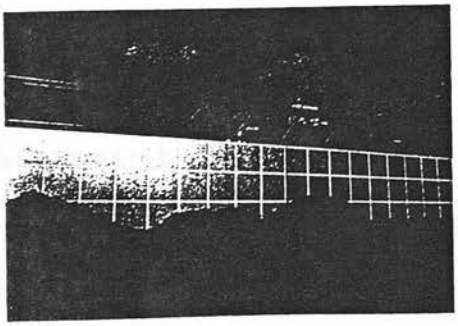












CASE STUDY No. 24

HOMEBASE, BRENTFORD

Consulting engineers: Ernest Green Partnership Ltd

Architects: Nicholas Grimshaw & Partners Ltd

1.0 Background

The architects were asked to design a building that was visually superior to the usual warehouse outlet. Nicholas Grimshaw and Partners used the structure to give the building its striking imagery. This is in keeping with the firm's philosophy and is appropriate to the warehouse tradition - when thinking of the great warehouses of the past it is the structure that comes to mind.

2.0 Architectural Concept

The concept was of an aircraft with the main central spine forming the fuselage, the mast forms the tail and the arc shaped roof sections, the wings (Fig 1).
All external columns were to appear as an aircraft undercarriage. Structurally, the scheme was to create a column-free space by means of a structural spine along the length of the building which was supported at an intermediate point by steel tension rods hung from a 33 m high tower. The tower structure supports an illuminated sign which can be seen from a considerable distance and stands clear above surrounding buildings when approached from the east.

The tower is placed at the entrance and the great beam passes

through it out to the tension rods in the car park; so shoppers driving up to the entrance to collect a partner with some heavy shopping, find themselves driving through the structure - a good porte cochere.

3.0 Structure

Study models prepared by the architects show the design input that went into the tower and the suspension structure. (Fig 5) Various arrangements which were sensible in engineering terms were made with visual considerations. The overhead structure here is similar in principle to the Oxford Ice Rink. It comprises a central spine beam that is cable-assisted.

The 95.7 m (319 ft) long central spine beam spans 75 m (250 ft) between the mast at the front of the building and the trestle at the rear, (Fig 12) and is supported at an intermediate point by steel tension rods hung from the 33 m (110- ft) high mast. Further tension rods take the load back to the tie-down anchor in front of the building. The spine girder is of tubular steel and the tie rods, (Fig 2A, 2B) 80mm diameter solid Macalloy bars.

Seven "wings" span from each side of the spine beam to V-shaped props on concrete elliptical plinths along the sides of the building. (Fig 1) In keeping with the aeronautical theme the wing sections are of tubular steel and all bracing are tie rods. The internal view of the final assembly is intended to resemble the wing construction of an early Caproni bi-plane. (Fig 8)

A barrel-vault rooflight runs along the spine beam, wrapping

around the "tail" end above the trestle. At both ends of the building, the rooflight is met by strips of vertical glazing, so that the building is cut by a "slice of light". (Fig 7, 10)

Each "wing" is clad with a sandwich construction of profiled aluminium sheeting and mineral wool insulation, following to the curve of the roof. Five rooflights are integrated with the cladding on each wing. At the end of each wing is a GRP "wing-tip" canopy . The roof is drained by gutters which run between the wings into "hoppers" between the wind-tips and by "wing-flap" gutters at each end of the building. (Fig 3, 4)

The structural linear trays, which also provide the internal wall finish, span vertically between an angle at the edge of the floor and an angle at the edge of the roof structure. The linear trays hold the mineral wool insulation and the external walls are clad with profiled aluminium sheeting.

The concept for the space is that it is to be totally flexible and open plan. Enclosures for short-term tenancies are temporary and they can be easily removed with little disruption to the store.

4.0 Erection

The erection was carried out in a most economical manner.

The spine beam was fabricated in seven sections with each section constructed in line with the previous and then transported to site. Both the 33m tower, and the 95m spine beam, were welded on

the ground.

The tower was erected then the spine beam passed through it in a lifting operation that took less than 8 hours.

The wing sections were fabricated on the ground and lifted into place, two or three in a day. (Fig 9) The tie rods had strain gauges fitted prior to erection and once the building was clad the turn buckles used to stress the rods to equal degrees.

5.0 Advantages/Disadvantages

The dramatic structure is impressive in image terms, but it also has significant functional advantages. At Brentford, Homebase has the ultimate in big sheds - one with no internal columns at all.

An internal column in the wrong place could waste many square metres of floor space by limiting the placing of aisles. the spacing of aisles varies according to the objects on sale, land changes with changes with changing consumer habits so it is not possible to make the centres of columns relate to aisle widths. The big span with no internal columns is one optimal solution for store operation.

The great disadvantage of big span spaces with clear structures is that subdivision is not a straightforward matter. There is no row of columns to take a wall, or even a simple purlin line to receive a partition. The partitions dividing off corner space are bad enough, but the requirement, late in the programme, that

one side of the building be leased to another tenant, resulted in a dividing wall along the length of the building, to the great detriment of the interior symmetry.

6.0 Roofing

Homebase's experience of other buildings inclined them against flat roofs and rainwater pipes located in the middle of floor plans. In this respect, the aesthetic effect of the "wings" meets the functional requirements of effective roof drainage. The geometry is such that it sheds water more effectively than the usual portal framed roof with its parapet gutters.

7.0 Criticisms

Internally, the structure that is so clear on the architects' sketches has heavy secondary members that confuse and rooflights that do little to help the geometrical order. One would have like the rooflights to be where the gutters are, so that the industrial structure of each wing would be apparent.

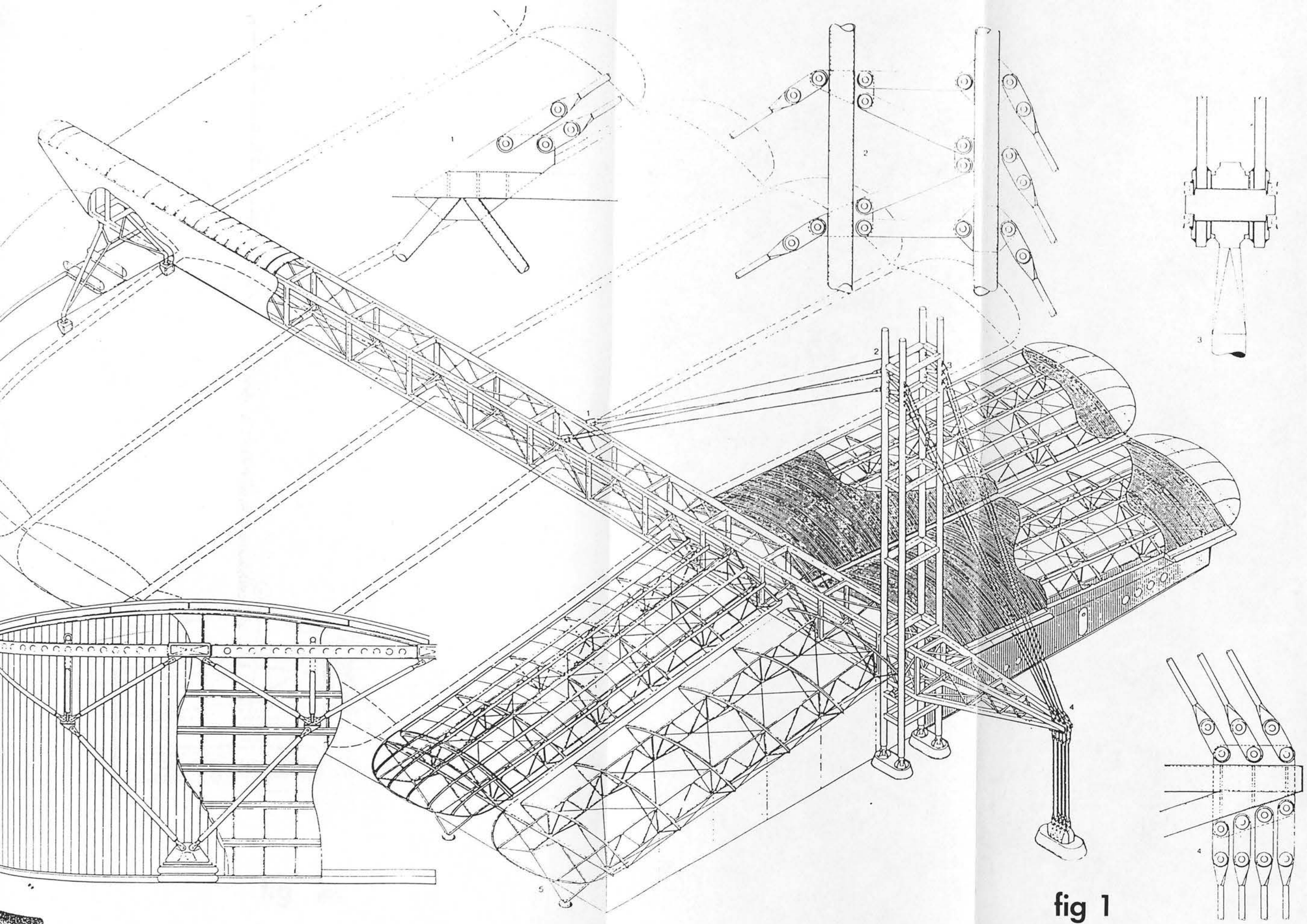
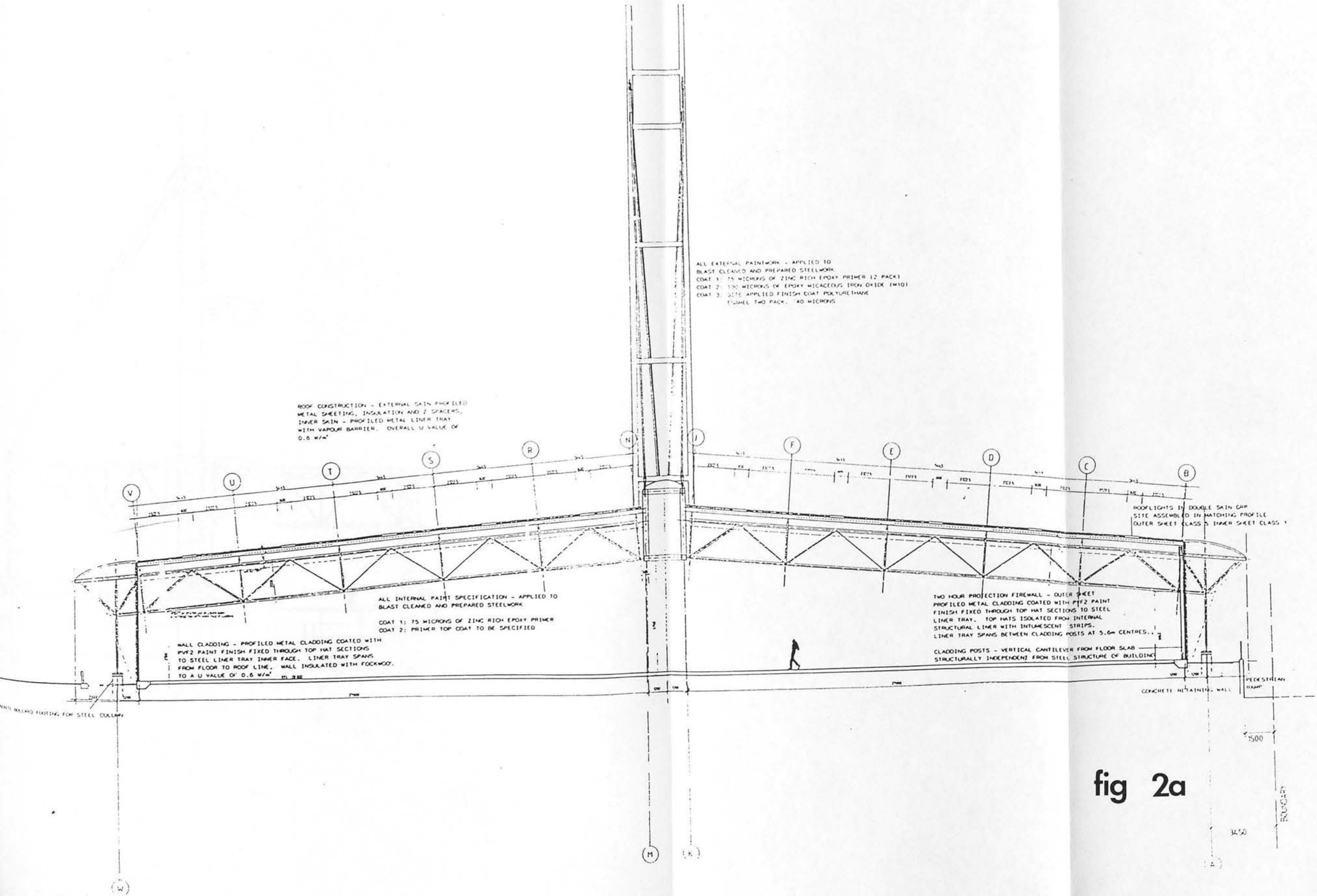


fig 1


ALL EXTERNAL PAINTWORK - APPLIED TO
BLAST CLEANED AND PREPARED STEELWORK
COAT 1: 75 MICRONS OF ZINC RICH EPOXY PRIMER (2 PACK)
COAT 2: 100 MICRONS OF EPOXY MICACEOUS IRON OXIDE (MIO)
COAT 3: SITE APPLIED FINISH COAT POLYURETHANE
ENAMEL TWO PACK. 40 MICRONS
ROOF CONSTRUCTION - EXTERNAL SKIN PROFILED METAL SHEETING, INSULATION AND Z SPACERS. INNER SKIN - PROFILED METAL LINER TRAY WITH VAPOUR BARRIER. OVERALL U VALUE OF 0.6 W/m²
ROOFLIGHTS IN DOUBLE SKIN GRP
SITE ASSEMBLED IN MATCHING PROFILE
OUTER SHEET CLASS 5 INNER SHEET CLASS 1
ALL INTERNAL PAINT SPECIFICATION - APPLIED TO
BLAST CLEANED AND PREPARED STEELWORK
COAT 1: 75 MICRONS OF ZINC RICH EPOXY PRIMER
COAT 2: PRIMER TOP COAT TO BE SPECIFIED
WALL CLADDING - PROFILED METAL CLADDING COATED WITH PVF2 PAINT FINISH FIXED THROUGH TOP HAT SECTIONS TO STEEL LINER TRAY INNER FACE. LINER TRAY SPANS FROM FLOOR TO ROOF LINE. WALL INSULATED WITH ROCKWOOL TO A U VALUE OF 0.6 W/m²
TWO HOUR PROTECTION FIREWALL - OUTER SHEET PROFILED METAL CLADDING COATED WITH PVF2 PAINT FINISH FIXED THROUGH TOP HAT SECTIONS TO STEEL LINER TRAY. TOP HATS ISOLATED FROM INTERNAL STRUCTURAL LINER WITH INTUMESCENT STRIPS. LINER TRAY SPANS BETWEEN CLADDING POSTS AT 5.6m CENTRES.
CLADDING POSTS - VERTICAL CANTILEVER FROM FLOOR SLAB STRUCTURALLY INDEPENDENT FROM STEEL STRUCTURE OF BUILDING
CONCRETE RETAINING WALL
PEDESTRIAN RAMP
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D
C
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M
K
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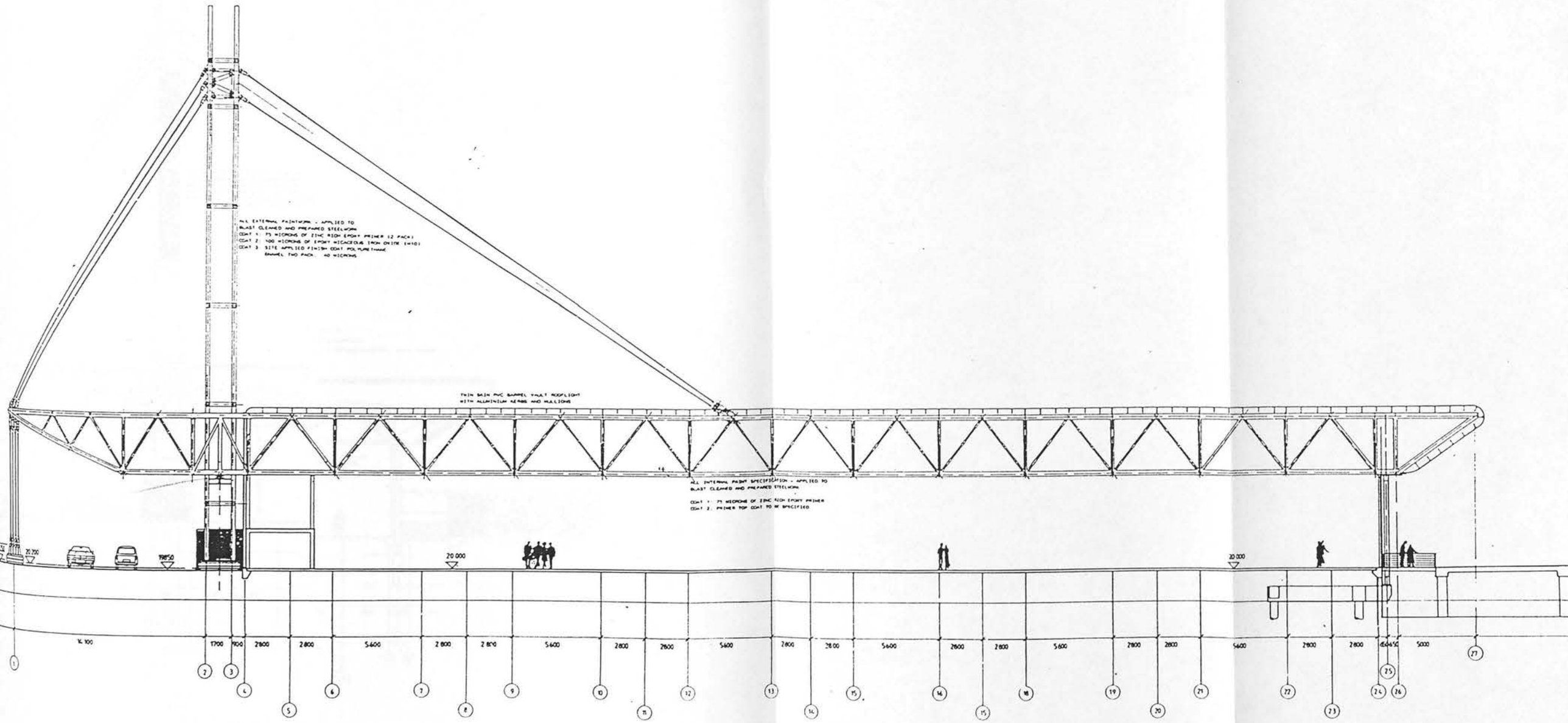
fig 2a


ALL EXTERNAL PAINTWORK - APPLIED TO
BLAST CLEANED AND PREPARED STEELWORK
COAT 1 75 MICRONS OF ZINC RICH EPOXY PRIMER (2 PACK)
COAT 2 100 MICRONS OF EPOXY MICACEOUS IRON OXIDE (1+10)
COAT 3 SITE APPLIED FINISH COAT POLYURETHANE
ENAMEL TWO PACK. 40 MICRONS
TWIN BAY PVC BARREL VAULT ROOFLIGHT
WITH ALUMINIUM KERBS AND MULLIONS
ALL INTERNAL PAINT SPECIFICATION - APPLIED TO
BLAST CLEANED AND PREPARED STEELWORK
COAT 1 75 MICRONS OF ZINC RICH EPOXY PRIMER
COAT 2 PRIMER TOP COAT TO BE SPECIFIED
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20 000
20 000
14 100
1700
900
2800
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5600
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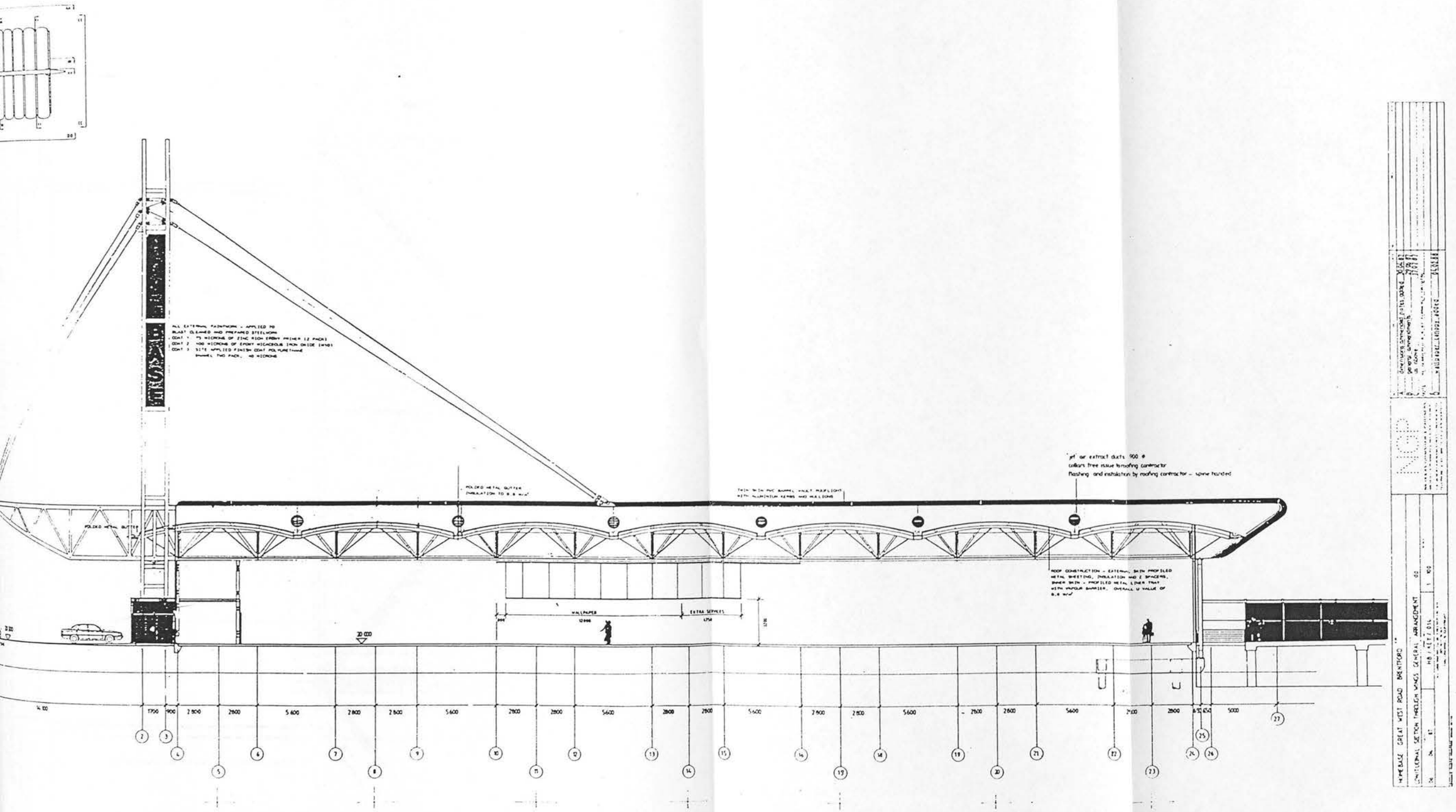
4640
5000

fig 2b


ALL EXTERNAL PAINTWORK - APPLIED TO
BLAST CLEANED AND PREPARED STEELWORK
COAT 1 75 MICRONS OF ZINC RICH EPOXY PRIMER (2 PACK)
COAT 2 100 MICRONS OF EPOXY MICACEOUS IRON OXIDE (MIO)
COAT 3 SIZE APPLIED FINISH COAT POLYURETHANE
ENAMEL TWO PACK, 40 MICRONS
POLIKO METAL GUTTER
INSULATION TO 0.6 W/m²
THIN WALL PVC BARREL VAULT ROOFLIGHT
WITH ALUMINIUM FRAMES AND MULLIONS
'pf' air extract ducts 900 ø
collars free issue to roofing contractor
flashing and installation by roofing contractor - spare handed
ROOF CONSTRUCTION - EXTERNAL SKIN PROFILED
METAL SHEETING, INSULATION AND Z SPACERS,
INNER SKIN - PROFILED METAL LINER TRAY
WITH VAPOUR BARRIER. OVERALL U VALUE OF
0.6 W/m²
WALLPAPER
12000
EXTRA SERVICES
1750
1250
20 000
4 100
1750
900
2800
2800
5600
2800
2800
5600
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HEMEL HEMPSTEAD
DRAWING NO
REVISION
DRAWN BY
DATE
CHECKED BY
DATE
SCALE
1:100
DRAWING NO
REVISION
HEMEL HEMPSTEAD GREAT WEST ROAD BRENTFORD
CROSS SECTION THROUGH WINGS GENERAL ARRANGEMENT
HB FACTORY DEV.
Arup

fig 3

fig 4

RADIUS TOP OF CLADDING 20.100m nom
RADIUS TOP OF PURLINS 20.000m
RADIUS TOP OF CHANNEL 19.787m

MIN VERTICAL DISTANCE FROM FFL 3700 AT WALL
MAX 6150 AT SPINE

PAIRS OF CLADDING POSTS ON FIRE WALL ONLY SUPPORTING TEE AT 25.350

WALL BETWEEN ANGLE AT BASE AND TEE IS SELF-SUPPORTING

SUPPORTING STEELWORK TO EAST WALL IS PROTECTED TO TWO HOURS FIRE RESISTANCE

FIRE EXIT DOORS
STRUCTURAL OPENINGS
1774 × 2015 DOUBLE
934 × 2015 SINGLE

RWP

FFL 20.000

SECTION THROUGH WING (TYPICAL) 1:20 NB: section taken at incline to vertical

RADIUS CHANNEL IN WING-TIP 19.900m (at incline to vertical)
RADIUS ANGLE IN WING-TIP 20.000m (at incline to vertical)
RADIUS ANGLE AT TOP OF WALL 19.977m (vertical)

PAIRS OF CLADDING POSTS ON EAST WALL ONLY

RWP & HOPPER

GUTTER

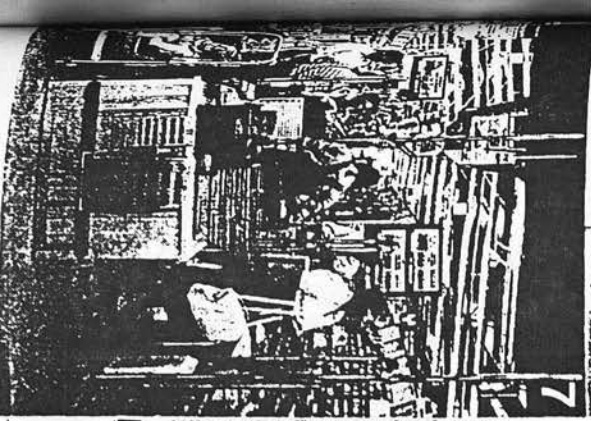
water bar

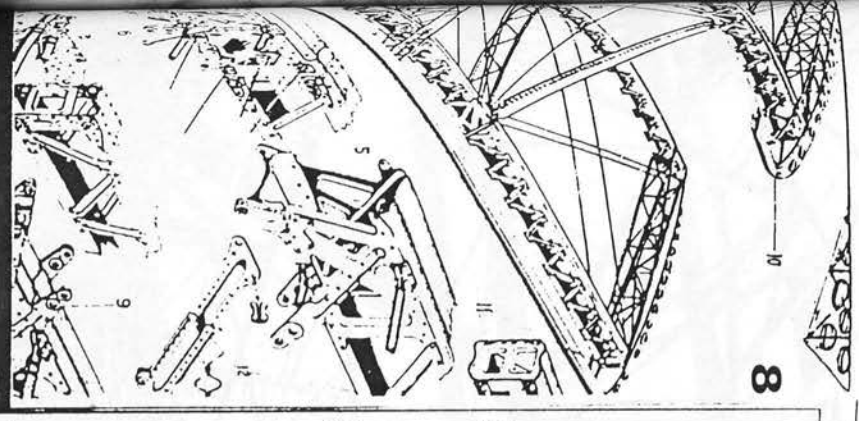
SECTION THROUGH WING ALONG CENTRELINE 1:20

SECTION THROUGH WING ALONG LATTICE 1:20 showing cladding posts

SECTION AT GUTTER 1:20

HOMEBASE GREAT WEST ROAD BRENTFORD
SECTIONS THROUGH WING, WING-TIP, GUTTER



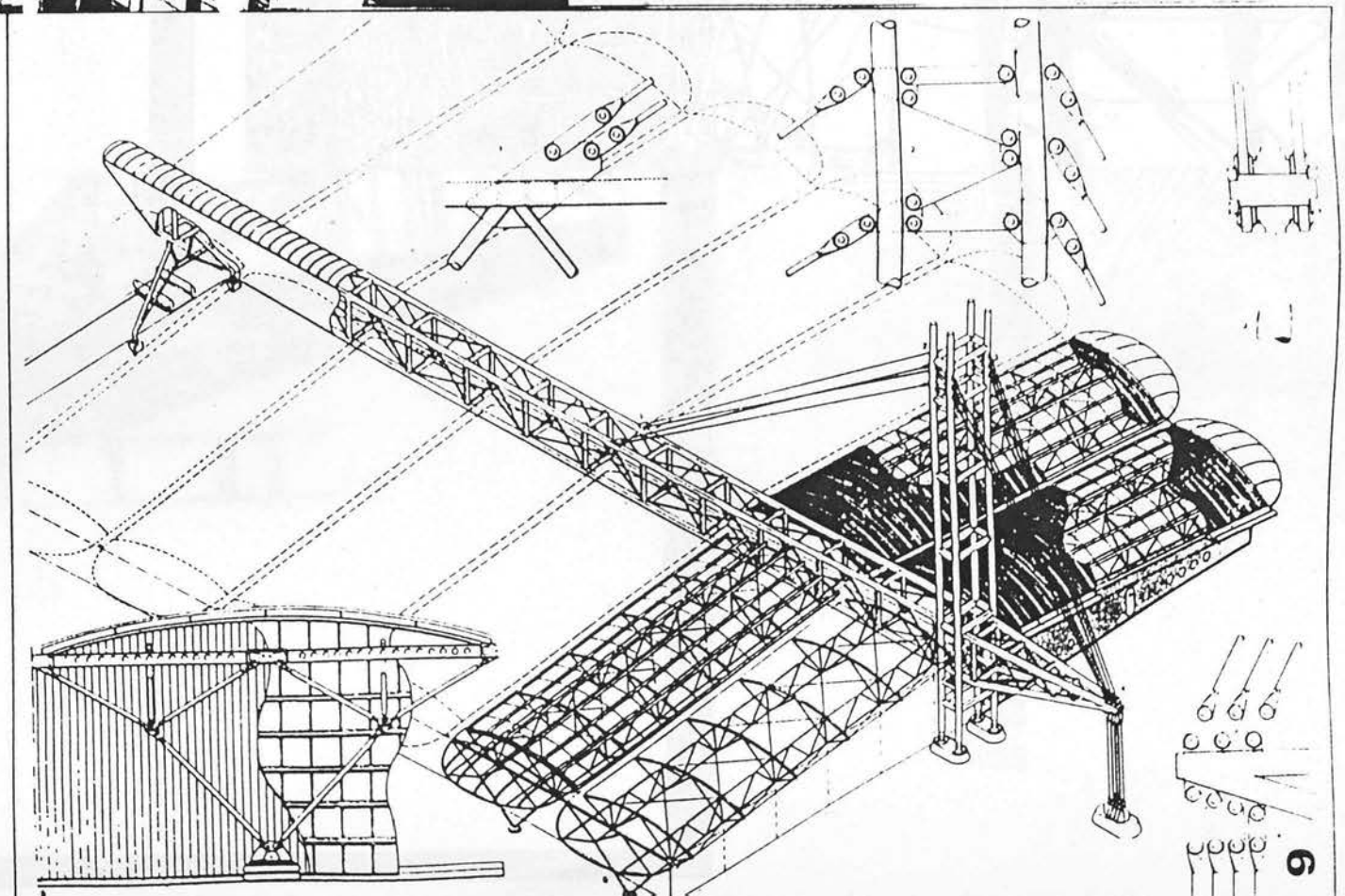


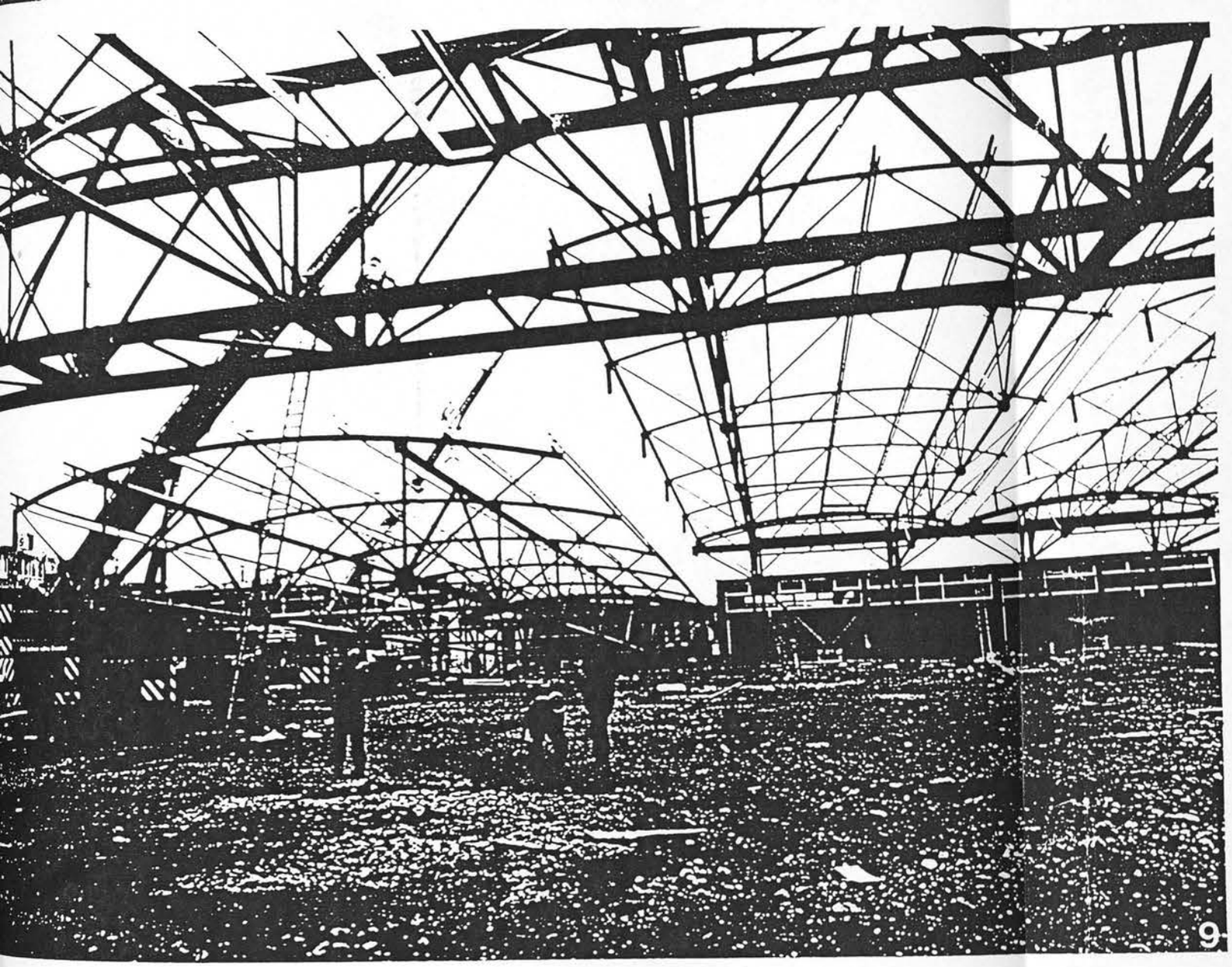
5 Wing of a Caproni biplane: a drawing much admired by the architects while they were designing Homebase.

6 Isometric drawings showing the steel structure and some of the aluminium cladding. The way that the form of the roof breaks down into a series of 'wings' is clear.

7 Interior: the main beam dominates the structure on the inside and is emphasised by the translucent roof. Fluorescent strip lights placed on the diagonal add to its impact. The photograph shows the pleasant warehouse feel of the interior.

5 Structure studies. The design team investigated various alternatives for the suspended structure before making the final choice. The solution is as the model on the right but with the tower modified to give more open framing.





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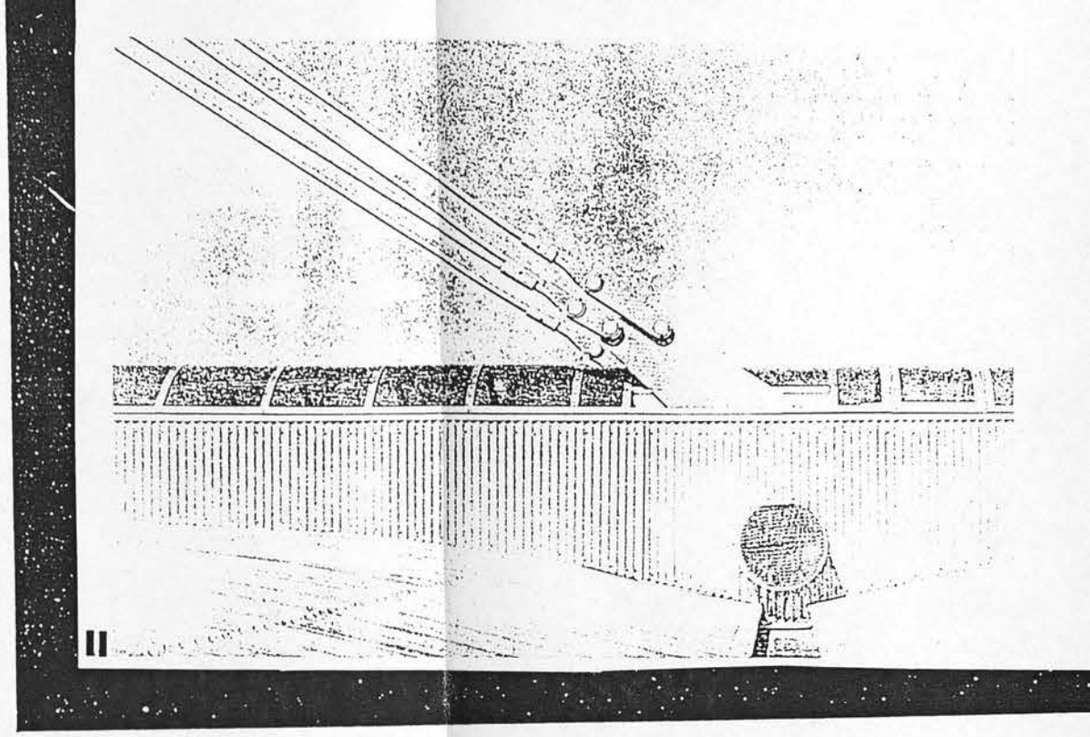

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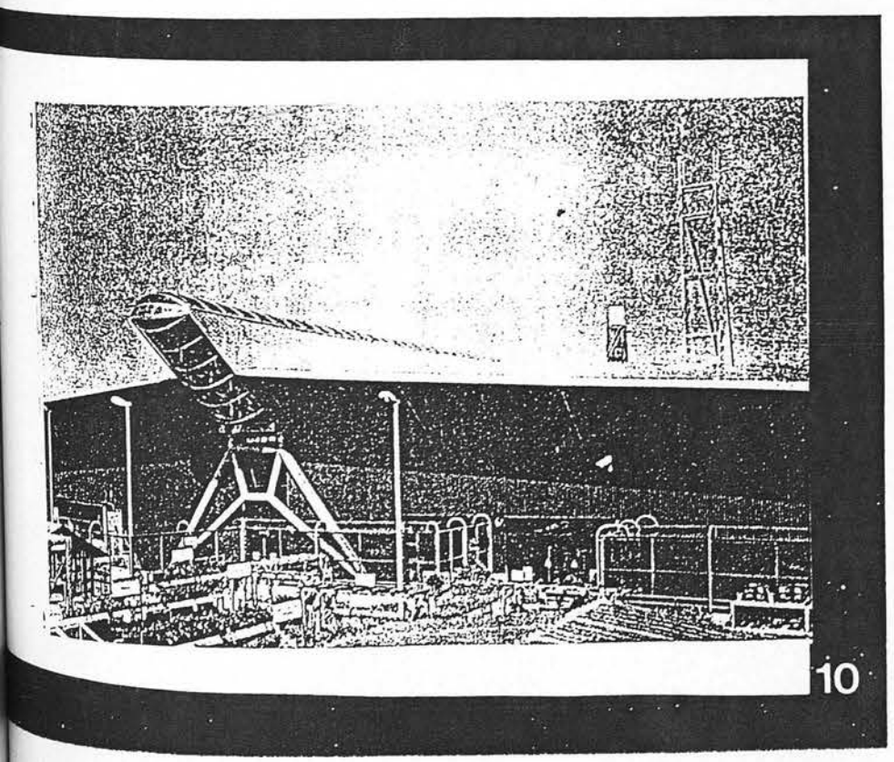

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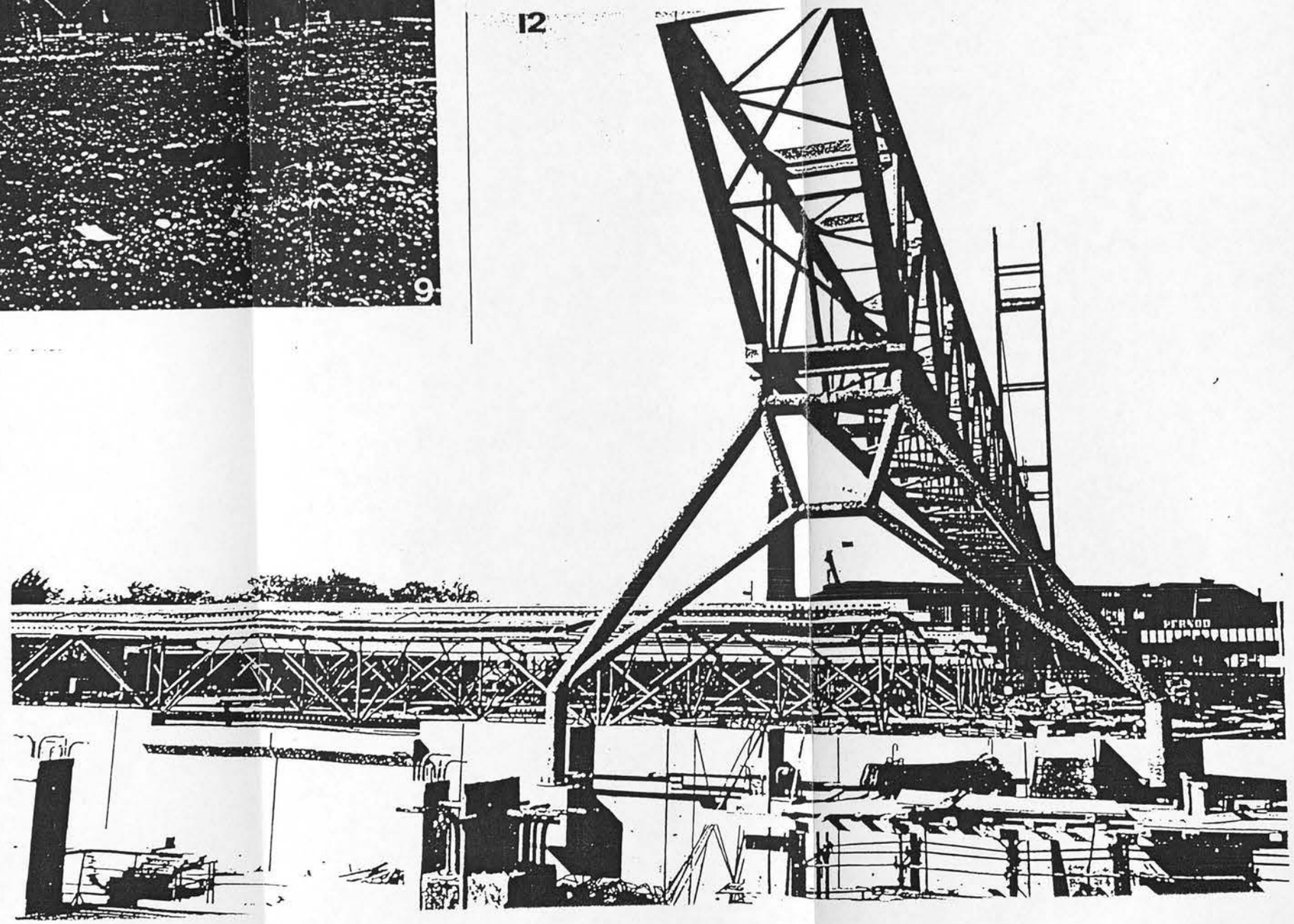
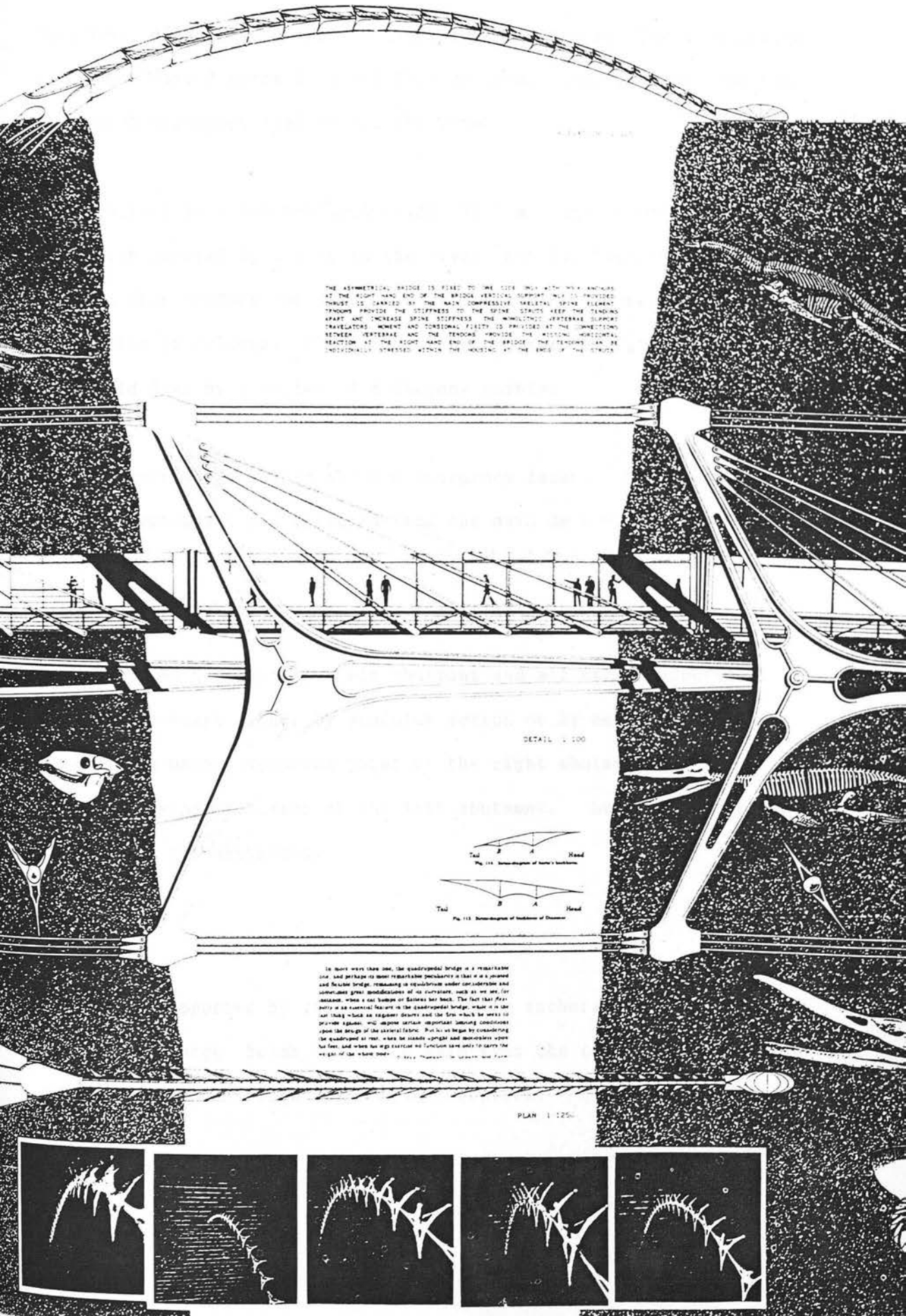


GROUP 3

case study 25

(Marks / Barfield)

Bridge of the Future



CASE STUDY 26: THE AUSTERLITZ BRIDGE ACROSS THE SEINE

(Competition entry by Richard Rogers and Ove Arup & Partners)

The Pont d'Austerlitz project links two of the main Paris mainline stations, Gare d'Austerlitz and Gare de Lyon, providing an important and useful transport link across the river.

The structure is a cable-stayed bridge with a single tower in the form of an arch carried by a pier in the river, not far from the left bank. The main span crosses the Seine to the right bank, where it is carried by a series of columns. The anchor span crosses the left bank, where it is held down by a series of anchorage points.

The deck carries six lanes and two emergency lanes. The footways and the rail system are cantilevered from the main deck on either side.

Articulation

The deck is anchored to the left abutment and all other supports allow relative movement either by pendulum action or by means of bearings. There is a major expansion joint at the right abutment and a minor joint, allowing rotation at the left abutment. Lateral forces are taken at the two abutments.

Deck

The deck is supported by cables connected to anchorages on both sides outside the edge beams. In the main span the cables are at 12 m centres and in the anchor span, at 4 m centres.

The main deck consists of a framework of steel beams, composite with a concrete slab. There is a box beam at each side and diagonal steel beams which span between anchorage positions at each side. The rail and the walkway are supported on brackets cantilevered out on each side.

Arch

The arch is a large steel tube whose form is determined to minimise bending moments.

The tube is stabilised against buckling in the direction of the main axis of the bridge by the main cables and against buckling in its own plane by braced cables which connect the arch to its supporting pier. At the upper ends the main cables are connected to the arch by anchorage connections.

The tower and the deck will be designed so that any one cable can be removed without over-stressing the structure.

Walkways

The walkways consist of prefabricated concrete units supported on brackets cantilevered from the sides of the main deck. The rail system would be carried in a similar way.

Design Development

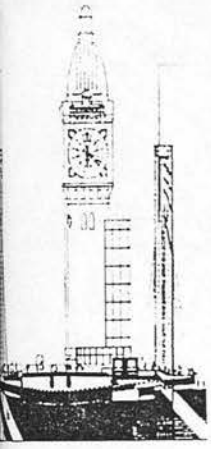
The sketch plans 1 to 8 indicate initial schemes with symmetrical layouts which finally developed into asymmetical arrangements. The forms of the pedestrian and vehicular bridges were initially separated

but finally integrated.

ELMANN

route, Jean Muller International, Sofresid, Bet

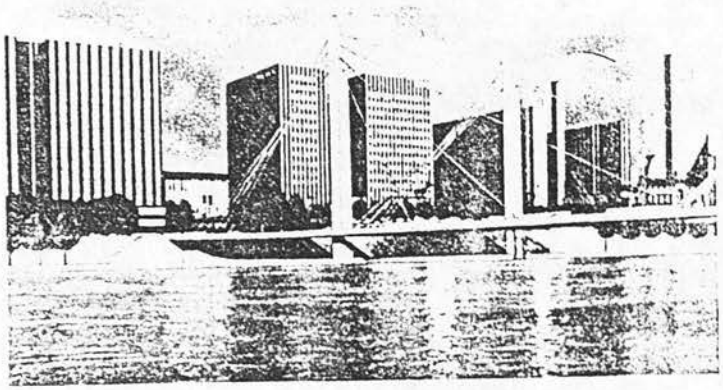
ier l'inconciliable. Etre à la fois ration et monument (le plus nt de Paris). Respecter un site oser avec la suite des ouvrages ts de rive à rive. Autour de ces s difficiles, l'agence Spielmann un projet ouvert et cadré sur la



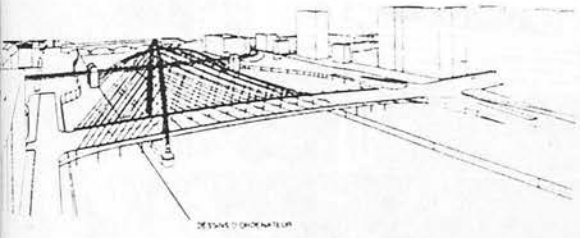
Le pont à haubans maintenu par deux pylônes ancrés sur la rive droite, permet de franchir sans appui le lit du fleuve et de dégager la rive gauche en vue de l'aménagement ultérieur des magasins généraux qui, pour l'instant, l'occupent. Le principe constructif est classique, mais les portées considérables : 161 m. Du coup, la structure s'affiche.

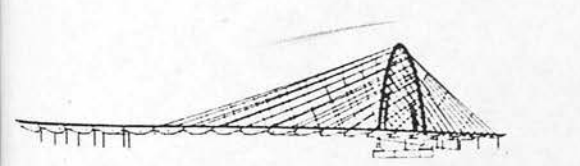
Simple ligne haubanée dans le paysage, l'ouvrage laisse le regard glisser d'amont en aval, d'autant que de larges trottoirs sur les quais et sur le pont appellent à la flânerie, les vues calmes sur l'eau et l'air. Dans l'axe de la traversée, du Sud vers le Nord, les pylônes distants et hauts de 40 m cadrent la tour de l'horloge de la gare de Lyon. Le choix des hauteurs renvoie à celles des bâtiments environnants et, plus loin, du nouveau ministère des Finances.

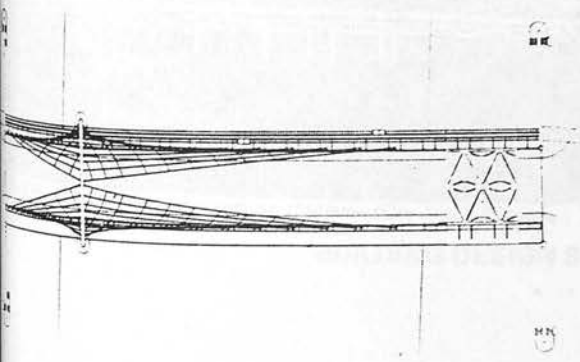
Tablier d'acier peint de couleur claire, garde-corps d'inox, pylônes, culées, escaliers et corniches lissés de marbre dessinent un pont à la modernité sage.



Cadrages sur la ville, haubanage puissant, modernité sage.
// The town framed in a powerful "ship's rigging", a knowing modernity.







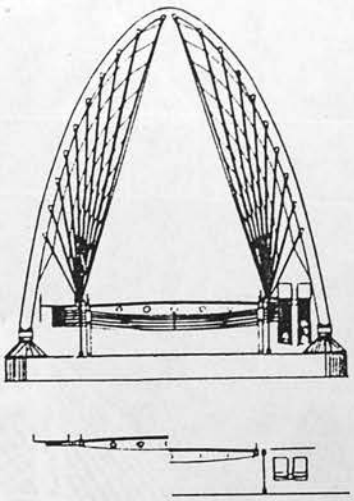
ROGERS

Ove Arup, Sofresid, Bet

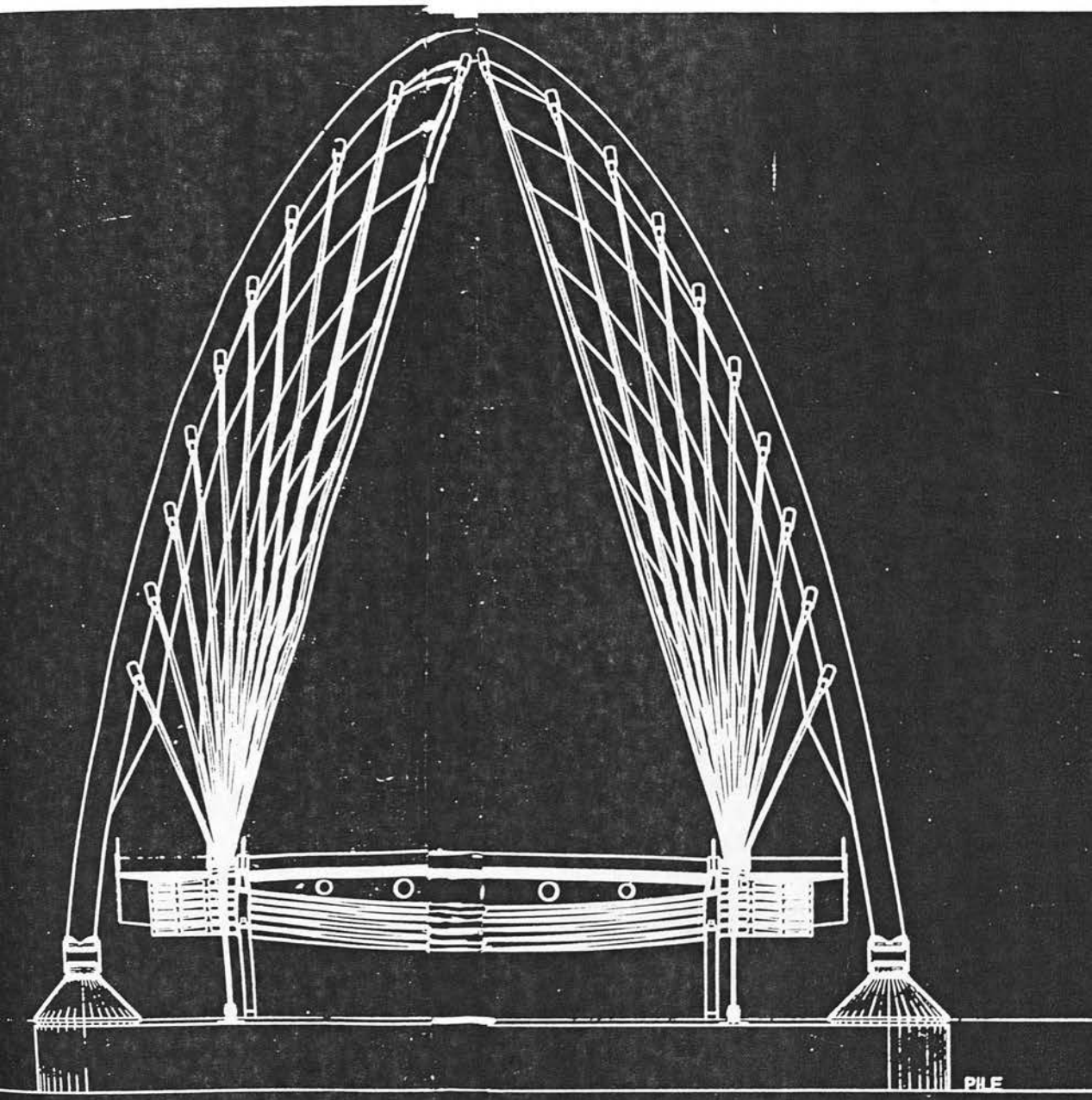
Projet mentionné par le jury, le pont de Richard Rogers adopte, sans le révolutionner, le principe du haubanage. Le choix répond à une volonté de légèreté. Un grand arc unique d'acier reçoit deux séries de câbles, tendus de chaque côté du tablier. Si l'impression de déjà vu domine, les cadrages sur Paris, les ponts tout proches et le fleuve, grâce à cette technique, ne sont encombrés ni vers l'amont, ni vers l'aval. Et de fait, le pont ne contraste pas avec les voisins, mais ajoute au leur un principe constructif.

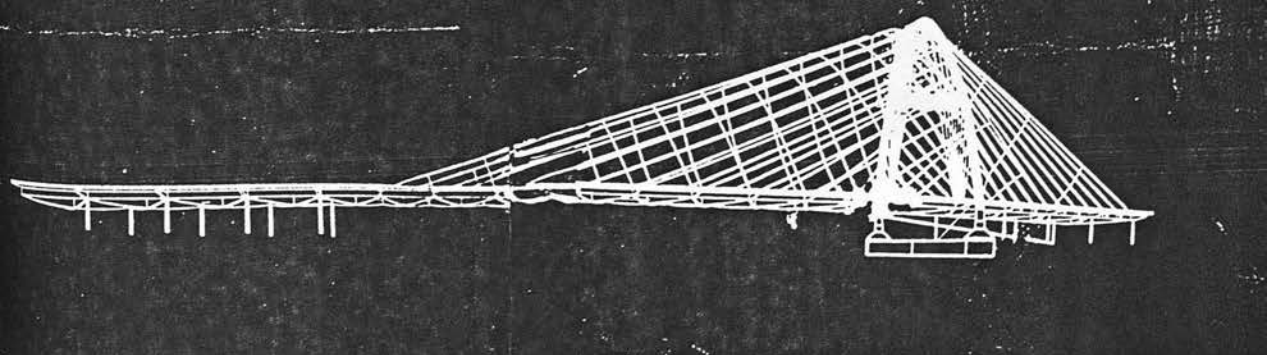
Alors que la structure trouve dans ses fonctions des aboutissements formels évidents, elle obéit aussi à une volonté manifeste : montrer, clairement, comment et par quoi un pont tient sur le vide qu'il franchit. D'ailleurs, quel que soit son lieu d'observation, le visiteur, l'automobiliste et le touriste en bateau-mouche perçoivent le grand arc solide comme un cadre dans lequel le paysage et les architectures apparaissent tour à tour. A cet égard, le campanile de la gare de Lyon, si proche de Big Ben, se transmute en signal, en phare. Point de mire.

Nappe de câbles en voûte réunis. Lisibilité.

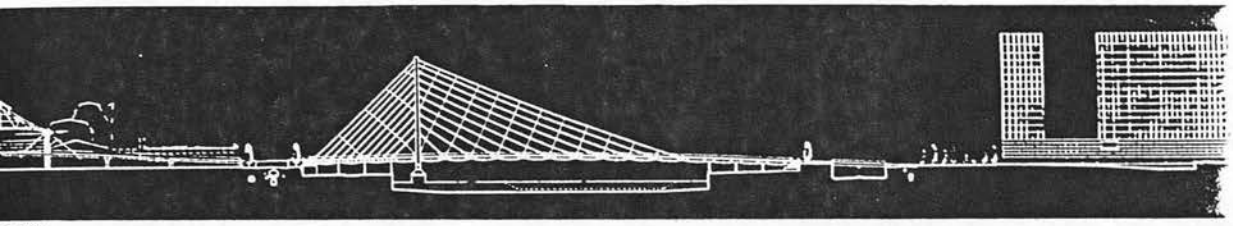


Avec un arc et une nappe de haubans, montrer comment un pont franchit l'espace.
// Showing how a bridge traverses space with an arch and a covering of cables.





...ion and perspective, generated by Caddraw (Ove Arup & Partners), modelling on Intergraph (Richard Rogers Partnership).



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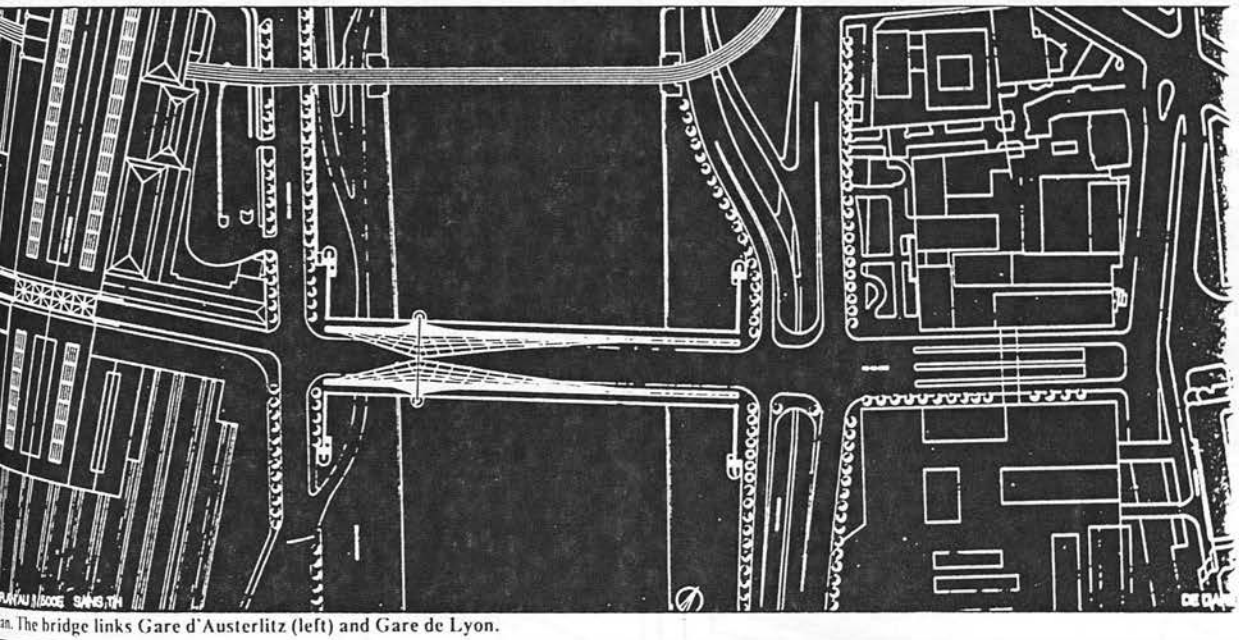
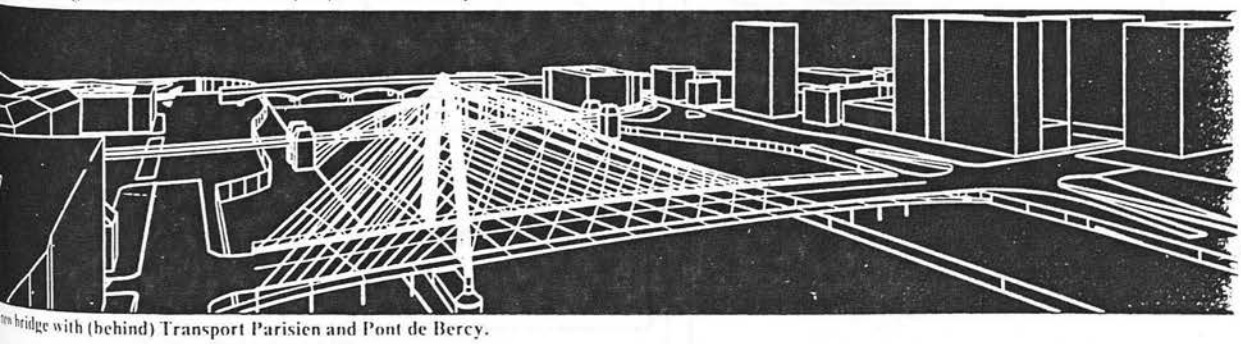
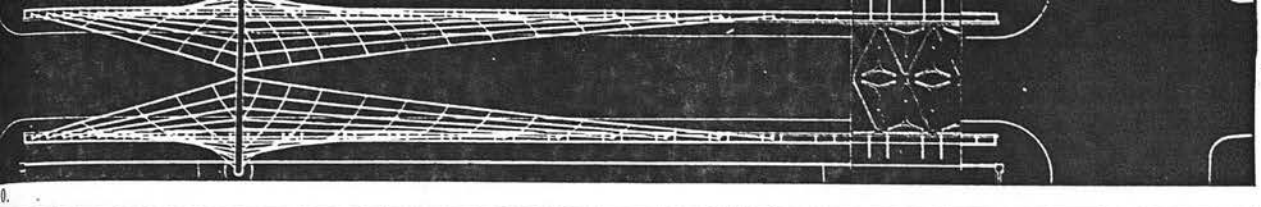


an. The bridge links Gare d'Austerlitz (left) and Gare de Lyon.



res bridge with (behind) Transport Parisien and Pont de Bercy.



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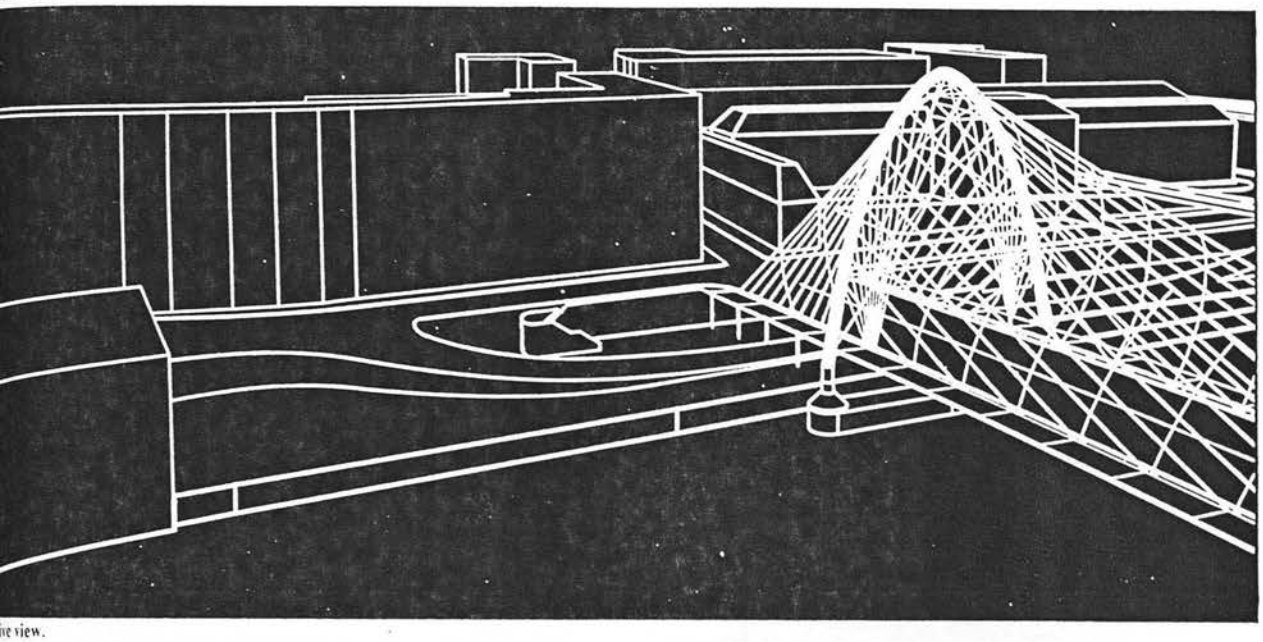
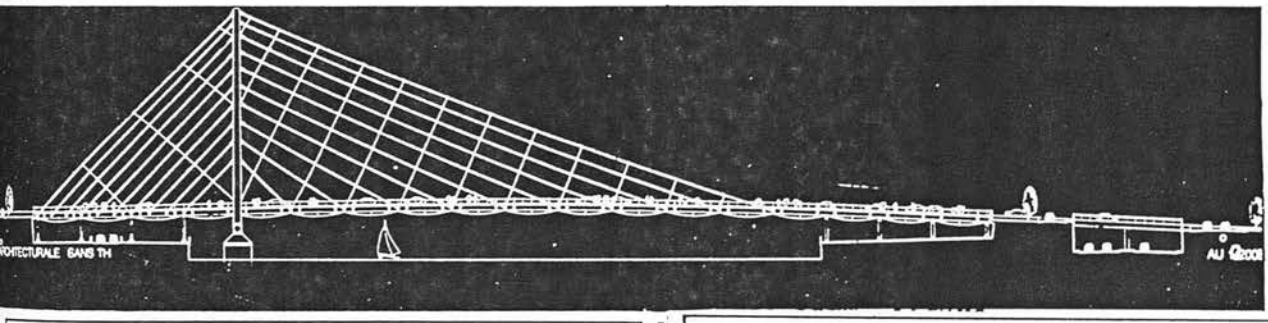
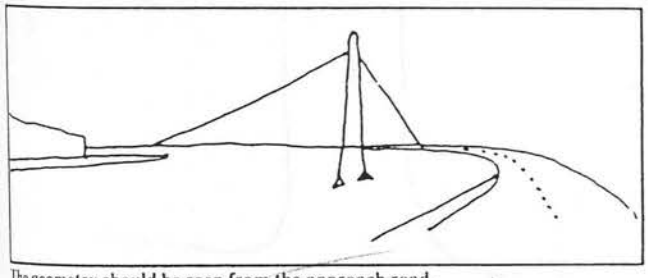
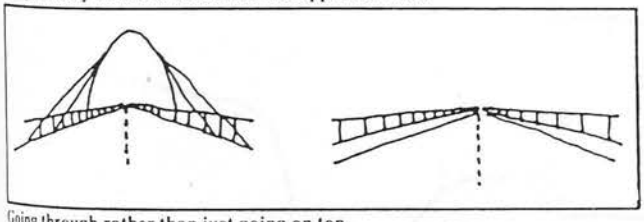


ne view.

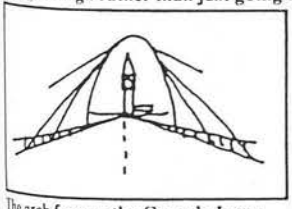




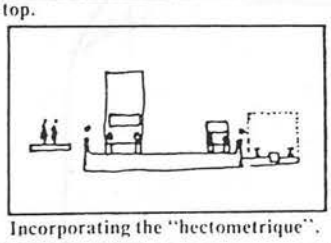
The geometry should be seen from the approach road.



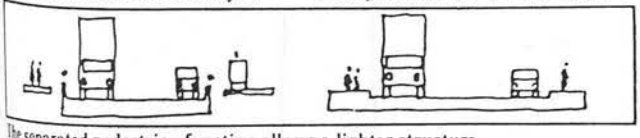
Going through rather than just going on top.



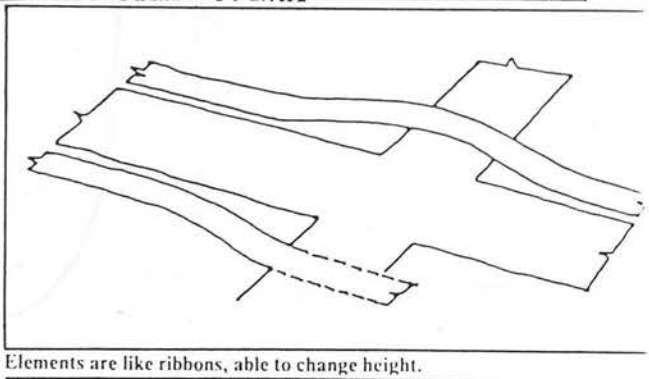
The arch frames the Gare de Lyon.



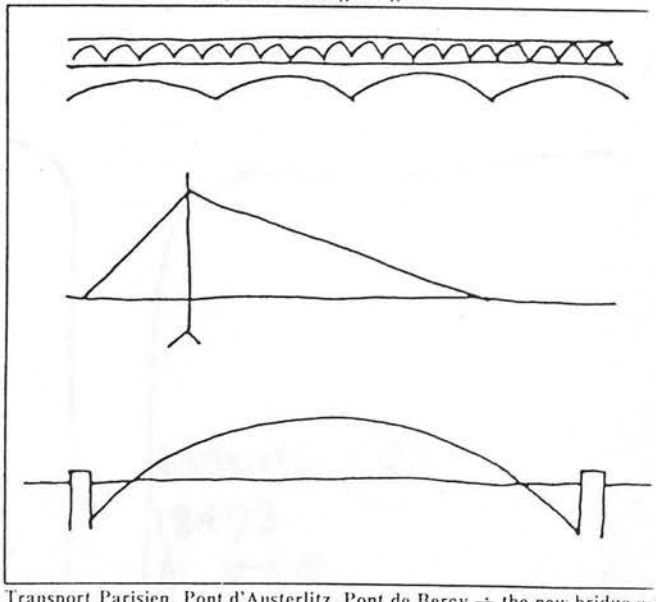
Incorporating the "hectometrique".



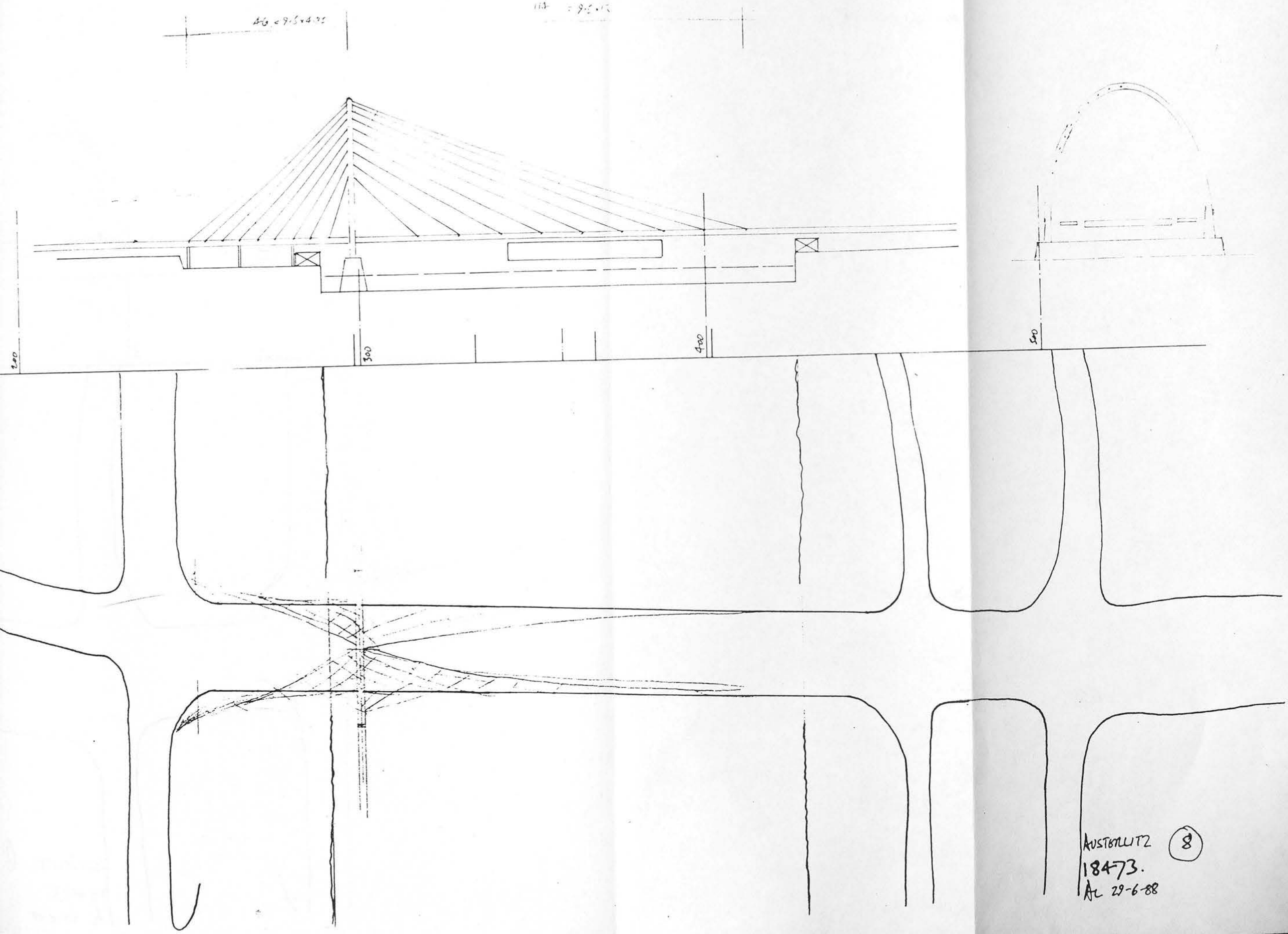
The separated pedestrian function allows a lighter structure.



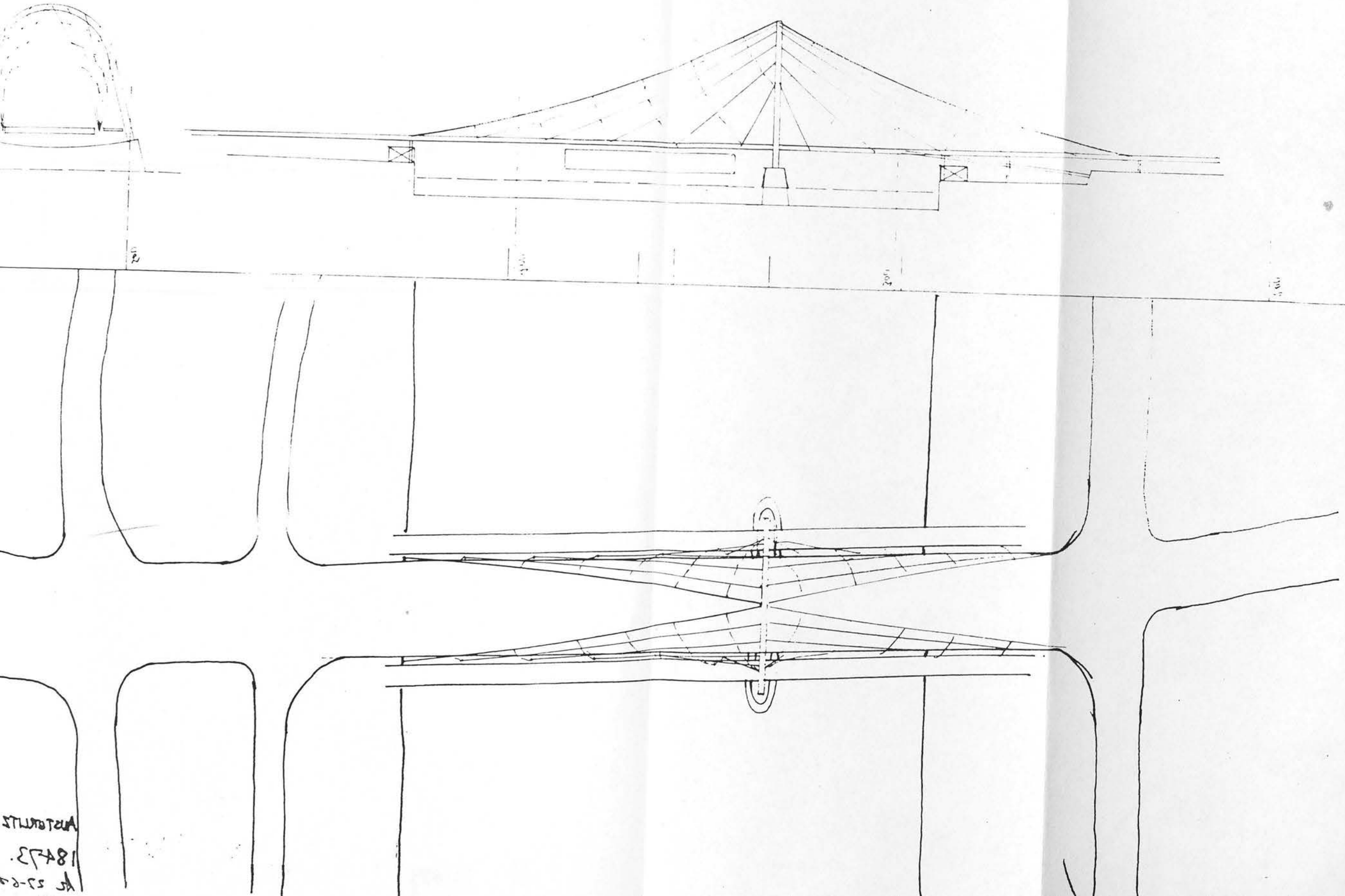
Elements are like ribbons, able to change height.

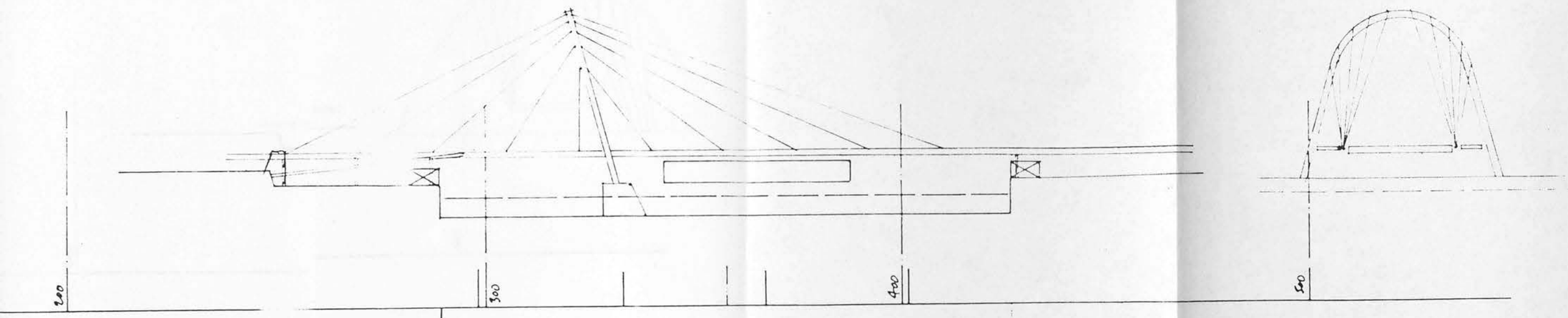


Transport Parisien, Pont d'Austerlitz, Pont de Bercy — the new bridge a
structural variety.

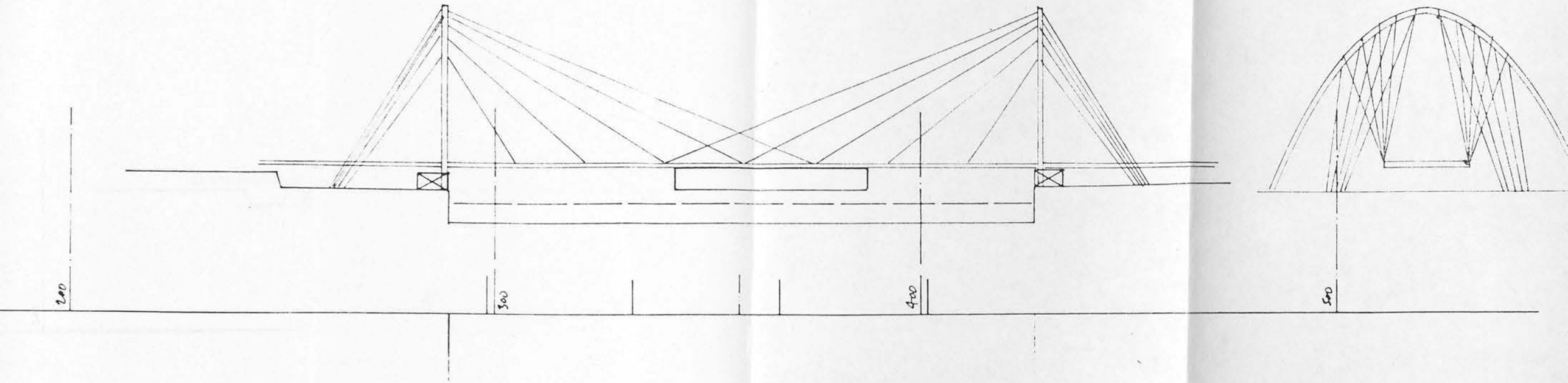


AUSTERLITZ (8)
18473.
AL 29-6-88





18473. (5)
R 77-1-20.



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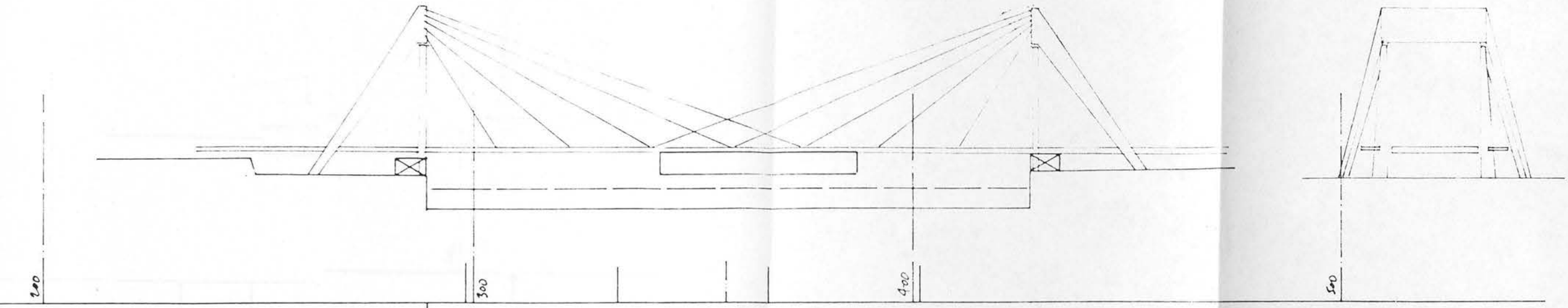
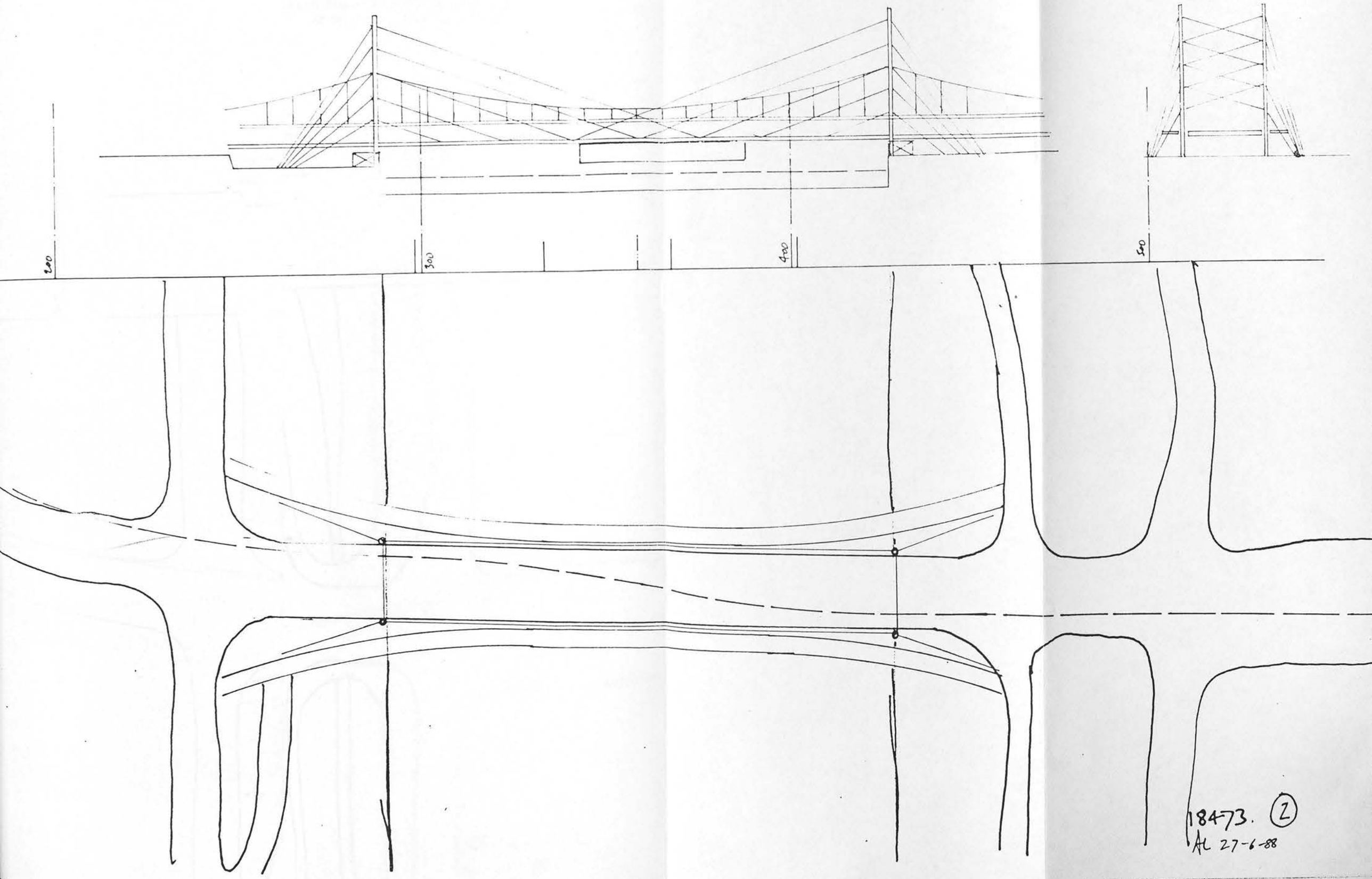
AL 27-6-88



18473. (3)
h 37 / 18



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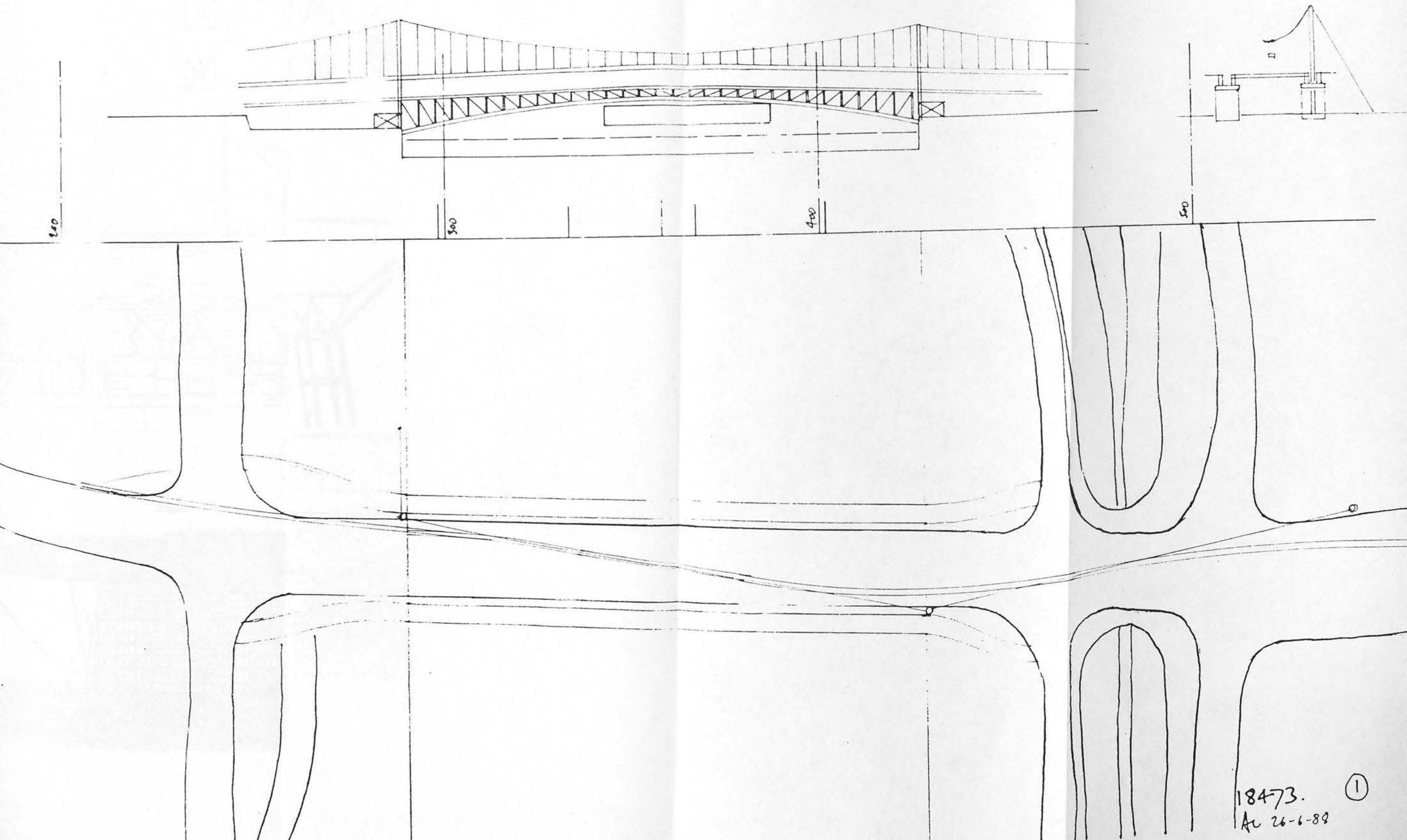
AL 27-6-88

Austerlitz Bridge Competition : Design Alternatives during inception stages.

Design Engineer, Angus Low, CAP
Architect , Richard Rogers



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AL 26-6-88

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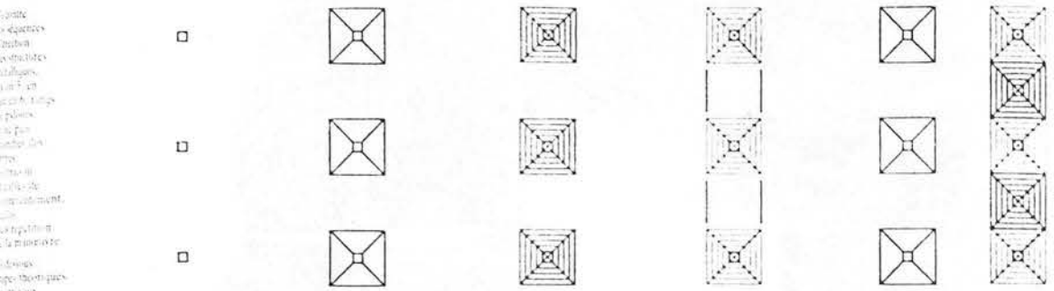
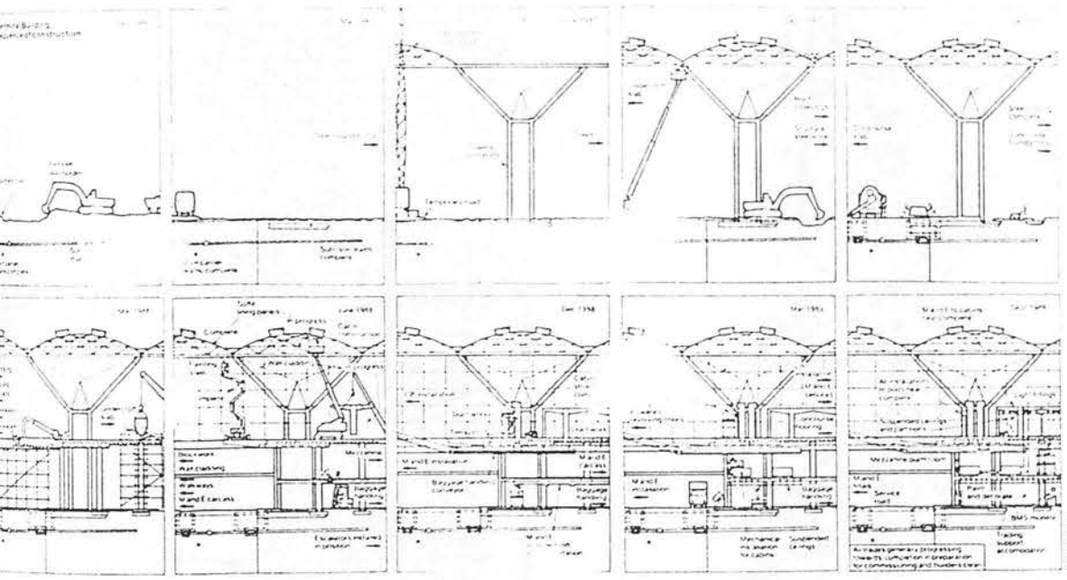
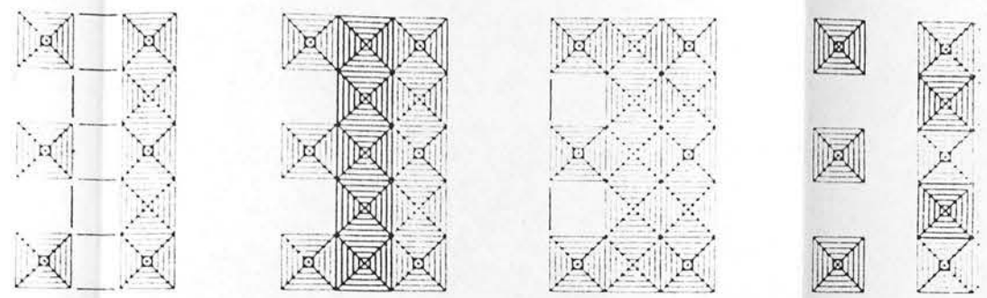
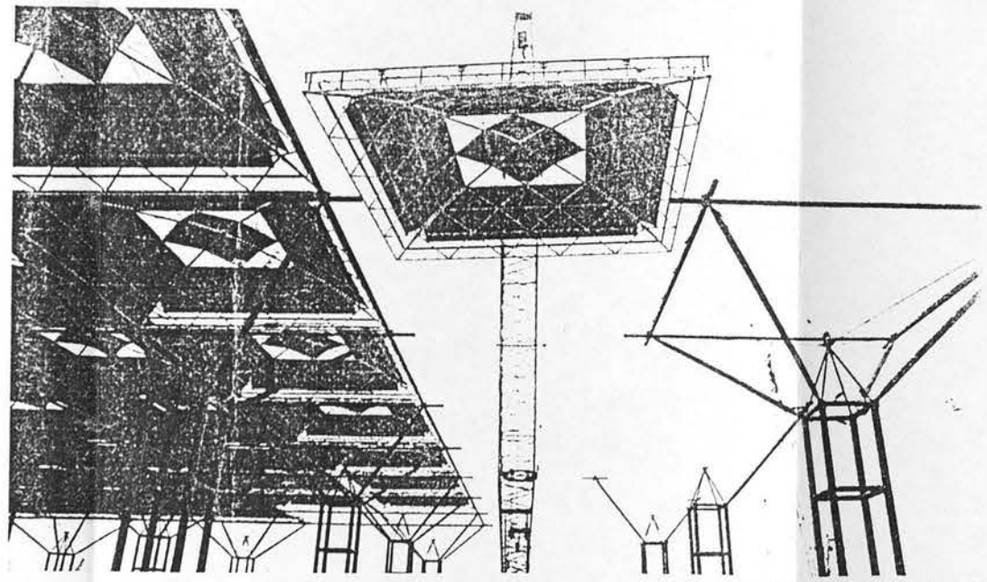
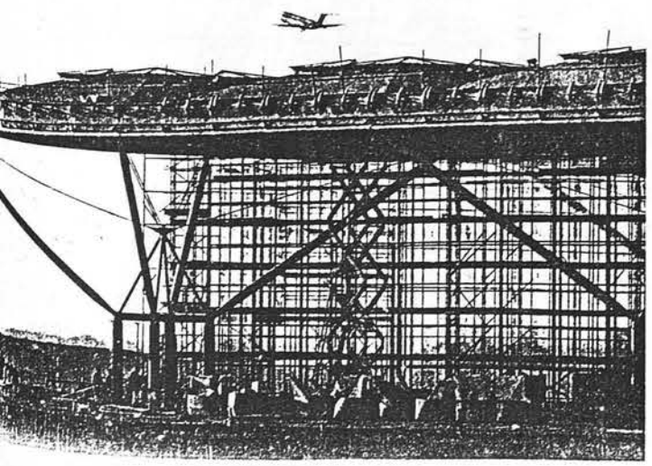


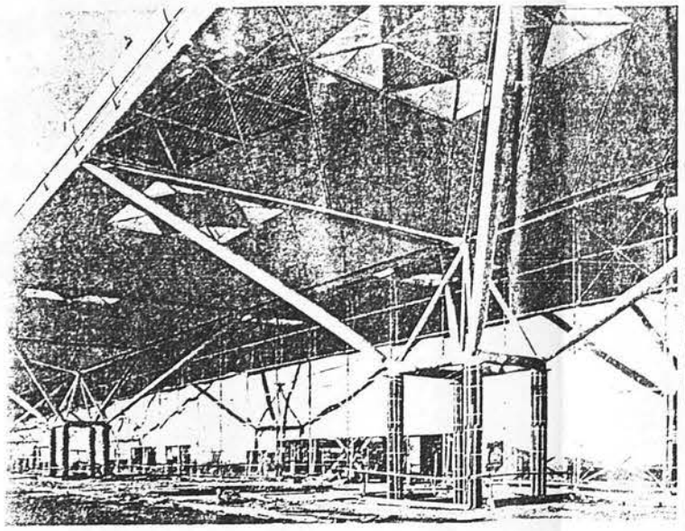


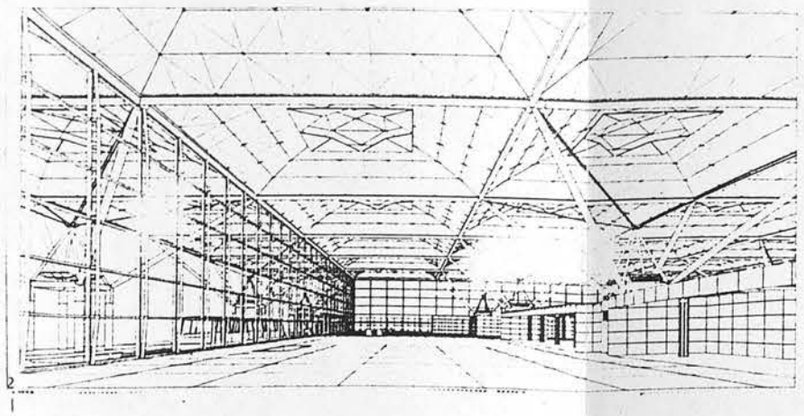




Pose d'une coupole sur un des pylo







case study 27

THE LIGHTS OF STANSTED

The transparency of the superstructure and the general horizontality of Norman Foster's design for the new terminal at Stansted, London's third airport, invokes an impression of aerial lightness. The first of the building's three levels is below ground. This makes it possible to put the main hall at ground level and keep the building at ground tree height, so that its presence does not overpower the rural nature of the site.

The base houses some parking, baggage handling, the building's technical services, provides warehouse space and zones for service vehicles. Short-term parking is located in front of the terminal and a rail link buts onto the S[illegible] edge of the building. From this [illegible] level, passengers reach the departure hall on the upper floor by lifts, escalators or ramps. After checking-in, automobile vehicles convey passengers to the departures satellites.

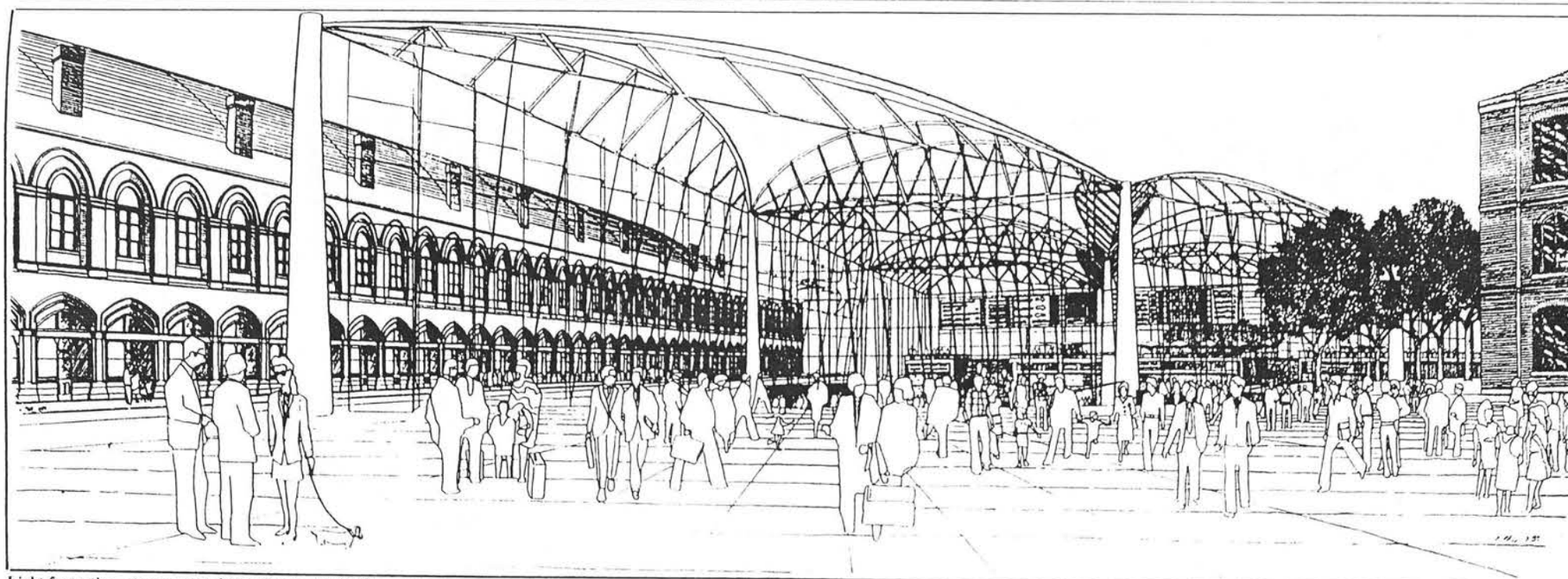
One of the attractions of the new terminal is that its transparence, particularly at the uppermost level, makes it easy to see the planes, landing, taking off or just standing around. This architectural demand for maximum transparence led Foster's research towards a compactness achieved by tangential spaces, which avoided corridors.

The tree-like structure frees the floor area of the level below. Ducting is housed in the structure's four risers. The roof itself is completely free of mechanical equipment.

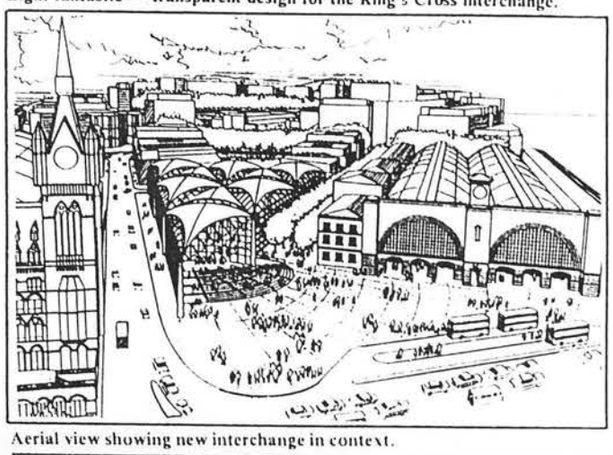
As with all Foster's work, the structure of the building reflects the interior spaces it creates. At Stansted the architecture is aerial, in the double sense that it is the ceiling that defines the volumes and that this volume is featherlight. There is also his usual attention to the interaction of natural and artificial lights, the interface between the two being his treatment of the glazing. At Stansted, particular attention has been paid to the way the terminal looks at night. The upper level of the hall will be lit from within the roof; so that from outside, the light source is invisible.

Foster's spatial conception for the terminal puts views of vehicles (planes, shuttles, buses, cars, etc.) in contact with colourful crowds moving at differing speeds. At every moment there is a consonance between mobile objects and the architecture.

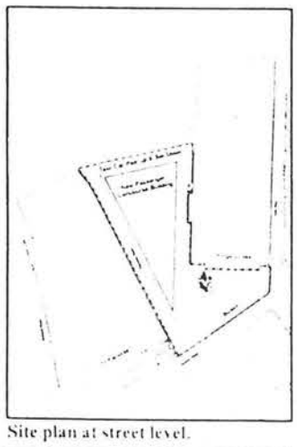
For Fosters's practice the terminal will represent a decade of involvement. Feasibility studies for London's third airport. Work began on site in October 1986, and is due to be completed by 1990.



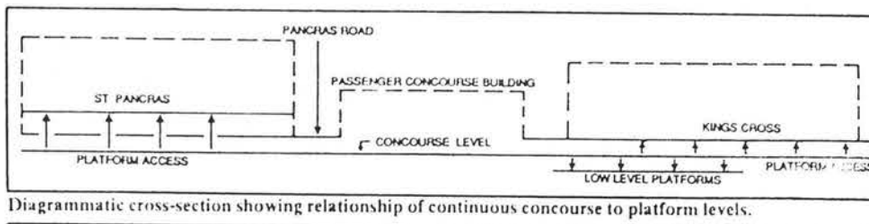
Light fantastic — transparent design for the King's Cross interchange.



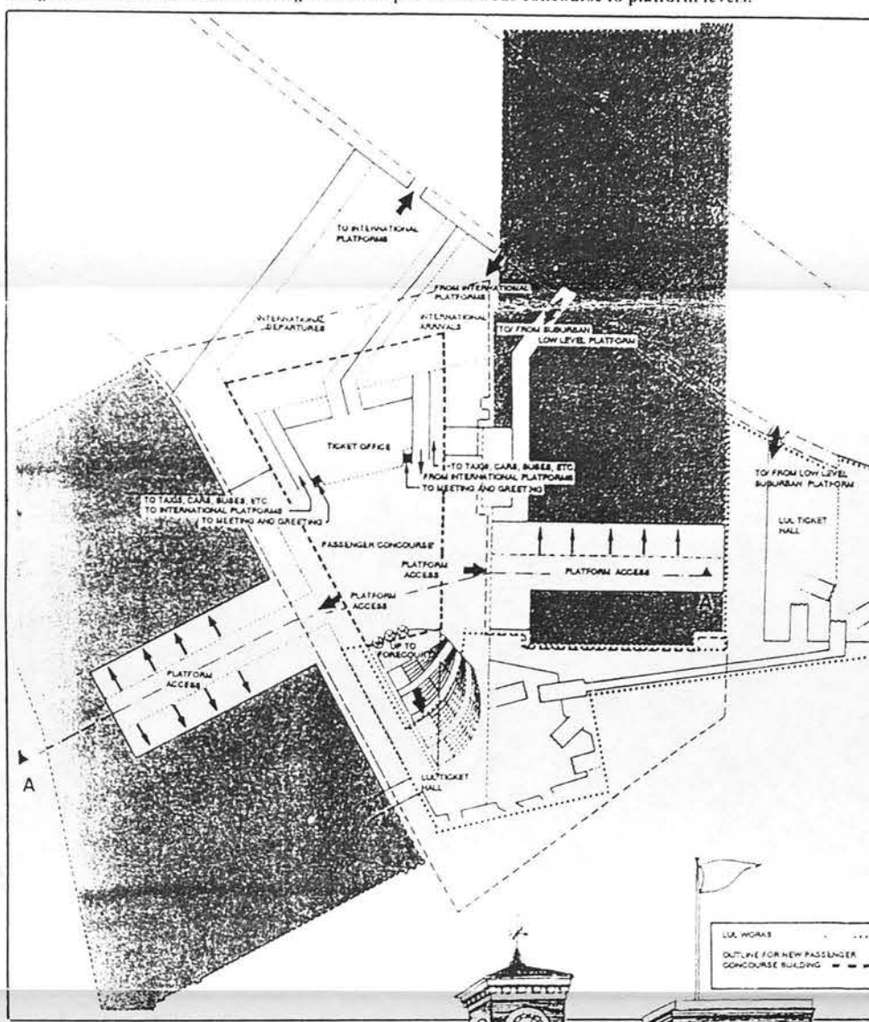
Aerial view showing new interchange in context.



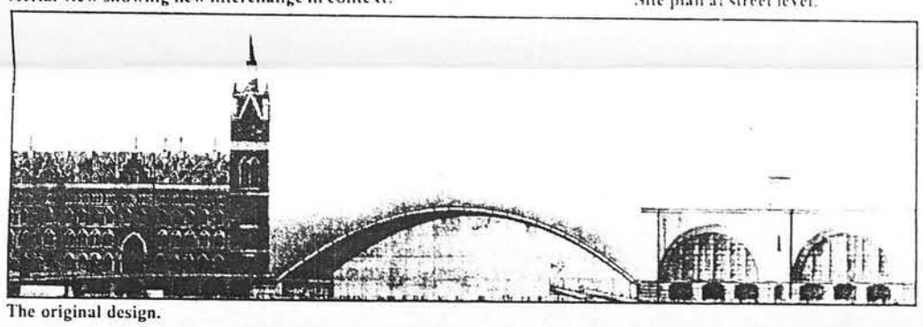
Site plan at street level.



Diagrammatic cross-section showing relationship of continuous concourse to platform levels.



Site plan at concourse level.



The original design.

Travelling light

FOSTER Associates' designs for a new concourse between King's Cross and St Pancras stations in north London have been unveiled, following submission of an outline planning application to Camden council.

The concourse has changed considerably since the unveiling of preliminary proposals when Foster Associates were appointed masterplanners for the King's Cross Goods Yard complex.

The first design dominated the low-rise King's Cross frontage, and filled entirely the gap between the stations. The latest version is minimal but still imposing, totalling as it does 10,000sq m.

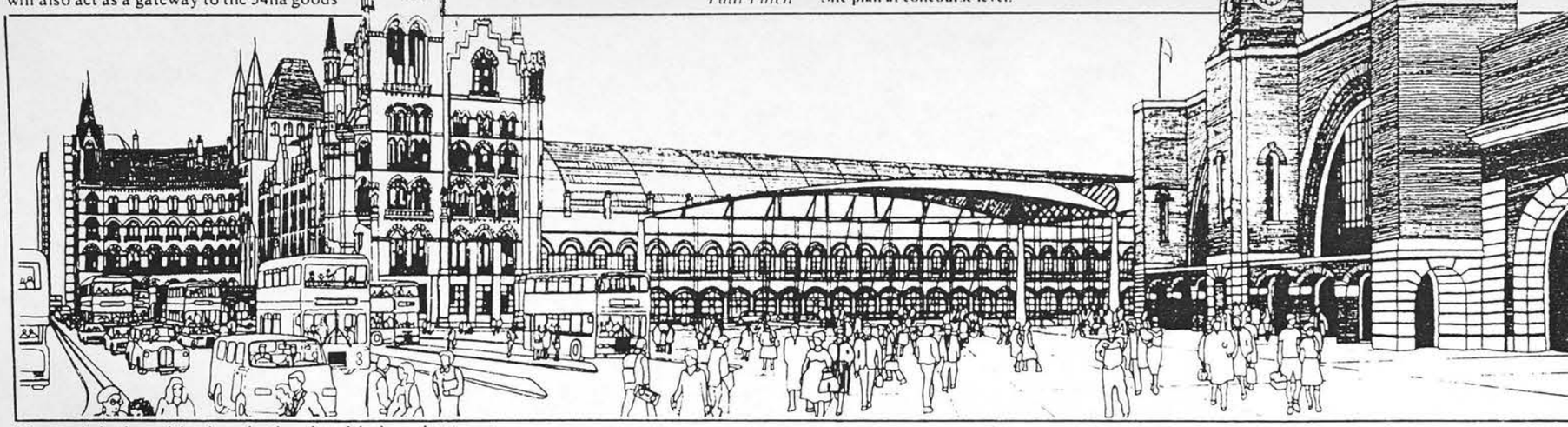
The result, described by British Rail as Europe's most comprehensive interchange for domestic and international travellers, will also act as a gateway to the 54ha goods yard site to the north being developed by Rosehaugh Stanhope.

Subject to legislation now before Parliament, the complex would give access not only to the two existing stations, and the Underground, but to international and cross-Thames trains.

The brief for Foster's has been to provide "elegant simplicity". While the proposal involves the demolition of the Great Northern Hotel, it otherwise respects the grade I-listed train sheds and Midland Hotel. The roof of the new building would be no higher than the cornice line on the St Pancras train shed side elevation.

The steel and glass aesthetic adapted by Foster's for the Stansted air terminal, now nearing completion, has been incorporated at King's Cross, where the design is intended to give the freest possible movement to very large numbers of commuters and other passengers.

Paul Finch



The new design is much less intrusive than the original grand statement.

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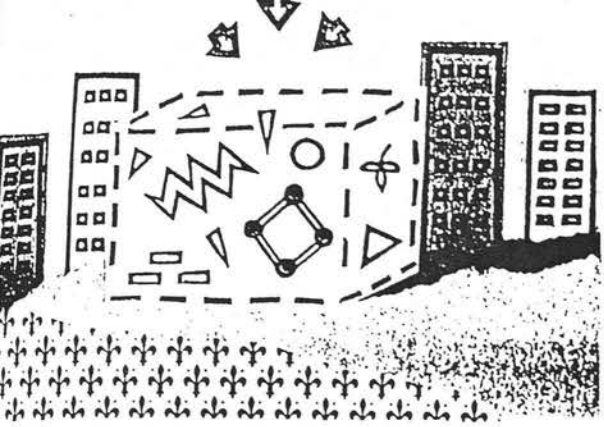
Enter 10 ON CARD ENQUIRY CARD



case study 29

In 1990, Glasgow will become European City of Culture, joining ranks with the other cities and capitals so far awarded this honour: Athens (1985), Florence (1986), Amsterdam (1987), West Berlin (1988) and Paris (1989). Glasgow's nomination was made by the government after successfully outbidding submissions from eight other cities in Britain. Glasgow has won this accolade because it was able to put the best and most convincing case, proving that it has international prestige, and a diverse and developing artistic life to complement one of the longest standing traditions of cultural provision in the world. It has also demonstrated how the arts and culture can reinvigorate a city undergoing regeneration and industrial change.

Glasgow District Council has taken an important lead in the build-up to 1990 by establishing a Festivals Unit to co-ordinate the plans, ambitions and activities of scores of the city's cultural organisations over the next three years. The city has already pledged substantial resources to the overall programme, as have Strathclyde Regional Council, the Greater Glasgow Tourist Board and other public bodies. The programme will encompass a huge range of activities, performances, publications, exhibitions, conferences and festivities spanning many cultural interests. The Institution of Structural Engineers Eurodrome Competition is one of several major contributions proposed by local and national organisations which will make 1990 in Glasgow the focus for Britain's most significant and ambitious cultural event.

Structural engineering is an art and a science requiring both breadth of imagination and an understanding of cultural values. The European City of Culture year will encompass activities extending far beyond the narrow confines of 'The Arts.'

The Scottish Branch of the Institution of Structural Engineers and the 1990 Festivals Unit have a mutual ambition to provide an exciting structure to house major Festival events. The structure itself will be a dramatic feature and one of the highlights of European City of Culture year in Glasgow.

The Institution is promoting an international competition for the conceptual design of a covered place of assembly of 4000m², or about the size of a small stadium. The winning design will be imaginative, innovative and represent the future direction for such structures. The building will be constructed in 1989 ready to house events from 1 January 1990 and will be dismantled at the end of that year with the possibility of re-erection elsewhere.

The site chosen by Glasgow District Council in collaboration with Strathclyde Regional Council is the former High Street Goods Station at the junction of Duke Street and High Street. This is a central location in an area of Glasgow currently undergoing rapid transformation and has excellent public transport access.

The total cost of the building is estimated at £2 million which it is hoped to raise by means of sponsorship. The natural prominence of the building should form an ideal opportunity for a major sponsor wishing to make a notable contribution to the 1990 Festival.

GLASGOW 1990
EUROPEAN CITY OF CULTURE

THE

GLASGOW

EURODROME

1990

ITION CONDITIONS

ER

stitution of Structural Engineers invites submissions for a Conceptual competition to design a structure for a place of assembly during the an City of Culture year of 1990 in the City of Glasgow. The ition is in one stage.

IVE

jective is to provide a design which shows the contribution of ural Engineering to the Built Environment in displaying both the art ience of engineering. It is hoped that the design will be transformed building to be used as a place of assembly during the year 1990 and demonstrate the objective of the competition in the real world.

INNING DESIGN

inal selection of the winning design will be by the promoter. The sors will select a short list of the three best designs submitted eport with an assessment to the promoter. The promoter will select a r and runner-up and make an award. The promoter may wish to interview uthors of the three best designs.

BILITY

ompetition is open to all Structural Engineers who are Chartered eers registered in the United Kingdom (e.g. IStructE, ICE, IMechE etc) o have an equivalent European qualification.

of Structural Engineers may make submissions endorsed by an individual tural Engineer.

ne entry is permitted by each Structural Engineer and firms making han one entry must comply with this requirement. The engineer is free llaborate with fellow engineers or other associated professions ed the entry is in his/her name.

IBILITY

fividual member of the Competition Committee of Scottish Branch of The ution of Structural Engineers nor the assessors shall be eligible to e or assist a competitor.

ORS

hite, the President of the Institution of Structural Engineers
ee from the District Council Festivals Unit
ice, an eminent Structural Engineer
Millan, Professor of Architecture : MacIntosh School
or

appointed as assessors to advise on the conduct of the competition. ll adjudicate on the designs and submit a final short list for the rs.

retary of the Institution of Structural Engineers has been appointed nister the competition. The manager may be contacted at:

e Institution of Structural Engineers
Upper Belgrave Street
ndon SW1X 8BH

235 4535.

12.00 monday
1st.

GLASGOW

EURODROME

1990

ITION BRIEF

S OBJECTIVES

onstrate the contribution of Structural Engineering to the built onment through a conceptual design competition for the design of a ing structure and its enclosure which can be demonstrated to be ional and capable of being constructed within the cost limits.

ngineer must apply skill, imagination and ingenuity to solving the ems set by this brief.

rief is to design a structure for a place of assembly in which events variety of forms can be held throughout 1990. The winning design d be capable of being constructed during 1989 and completed by December

winning entry, of innovative and imaginative design will reflect the it of the European City of Culture 1990 : GLASGOW.

GOW

gow is Scotland's largest city and is situated on the River Clyde. It raditionally one of the world's centres of engineering and related stries and its products have been sent throughout the world. Three ificant exhibitions have been held in the past in Glasgow. The 1901 gow International Exhibition, the 1911 Scottish Exhibition of History, and Industry and the 1938 Empire Exhibition all of which reflected the of the city in its time.

990 Glasgow, European City of Culture will once again become a festival . The Glasgow Eurodrome will become a public attraction, designed to modate a series of events throughout the festival year. It is intended uild the prize-winning design from this competition provided funds are ilable.

E

awing is provided showing the location and the extent of the site of the ding.

prevalent climatic conditions are separately provided in summary form attention is drawn to the combination of wind and rainfall which ires buildings to be carefully designed and detailed to ensure the usion of water. The geology of the site has been established in broad line and the structural design should be based on the subsoil profile rately provided.

proposals for the superstructure must show interaction with the subsoil.

site contours show the physical surface levels. The design of the ding structure should be based on the assumption of a formation level at 18.50 OD. Any ground irregularities will be assumed to be dealt with he provision of a level surface.

TS OF DESIGN BRIEF

object is to provide a conceptual design for a building structure.

design is to be limited to the loadbearing superstructure and its raction with the subsoil together with the external envelope of the ding.

planning of internal spaces and the servicing of the building may be idered in principle only and included in the description.

OF BUILDING

building will be known as "The Glasgow Eurodrome".

building structure will cater for three categories of activity in 1990 : mbly, Exhibition and Performance. The building will be dismantled in and may be re-erected elsewhere requiring a significant degree of portability.

THE GLASGOW EURODROME 1990

Entry by David Marks and Julia Barfield

CONCEPT

A super-light-weight arch structure restrained by secondary members all consisting of a fabricated kit of parts designed for ease of assembly and dis-assembly.

The form of the building is inspired by the elegance, hierarchy and inherent strength found in natural structures.

The generating geometry of the building is that it is a slice of a toroid.

A skeletal arch-spine spanning 110 metres is stiffened laterally by arch-ribs along its length at 5 metre centres.

The ribs are also laterally restrained by ties and the whole structure tied down against wind uplift.

The normal requirements of stiffness and rigidity are not considered here to be appropriate, even to the extent that settlement and spreading of the foundations beyond normal specifications will not impar the performance of this building.

FOUNDATIONS

The building is light-weight and temporary on a site which involves a relatively large amount of earth moving and importing of fill to produce a level area of reasonably sound formation. Resistance to the arch thrusts will be achieved by producing abutment zones filled during earthworks and founded on dynamically consolidated ground. These have the advantage of being more readily removed than mass bases or piling and pilecaps if they conflict with future development of the site.

The lightness of the structure will lead to uplift in the Glasgow wind and, again, the necessary earthworks and upfilling will be taken advantage of to place perimeter foundation pads at depth to add mass when these become ties.

SUPERSTRUCTURE

The spine consists of 21 vertebrae at 5 metre lengths. The main compression tube has a small pin casting at one end and a larger casting at the other which provides the connection between the struts, tube, ribs and can include provision for additional diagonal ties to increase the spine strength if required in the future, re-erected use.

Ties will be bars and struts are plate taper beams of varying length but identical base where they are welded to the hollow node castings. The unusual top tie is intended to facilitate erection when cantilever moments may arise and deal with negative bending in the spine under assymetrical loading.

THE SKIN

The proposed roof covering is tensioned fabric manufactured from a polyester membrane coated with PVC. If subsequent funding from a principal sponsor permits then alternative rigid covering can be provided either in its first use or when subsequently re-erected.

SERVICES ENGINEERING

Fresh air is introduced at the lower edges of the building. Natural stack-effect will draw air up and out through high level temperature sensitive self-powered opening glazed vents in the spine ridge.

Absorbant black netting will be suspended at a close distance below the roof membrane to reduce radiation reaching lower levels, and as this warms up, it will create more air flow around it and therefore increase the number of air changes.

Heating will be provided locally at low level according to the requirements of the particular activities and external climactic conditions.

Maximum use of natural daylighting and external awareness is achieved through the two fully glazed end walls and the spine ridge glazing.

Overhead services may be provided along the spine for lighting, PA systems , high level projection, overhead power and other suspended facilities.

Acoustic performance of the building can be modified by the introduction of foils suspended internally from the spine and ribs.

A black-out area may be provided under a mezzanine at the north-east end of the building.

COST

The overall form, volume and surface area of the building are cost-efficient in comparison to other single-span structures.

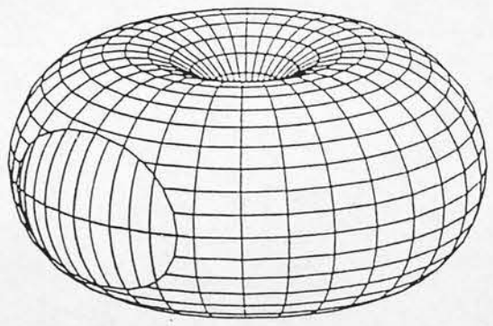
The landscaping is used to minimise the wall area, enhance the foundation performance and the building is clad in a relatively inexpensive but high performance fabric material.

VITAL STATISTICS

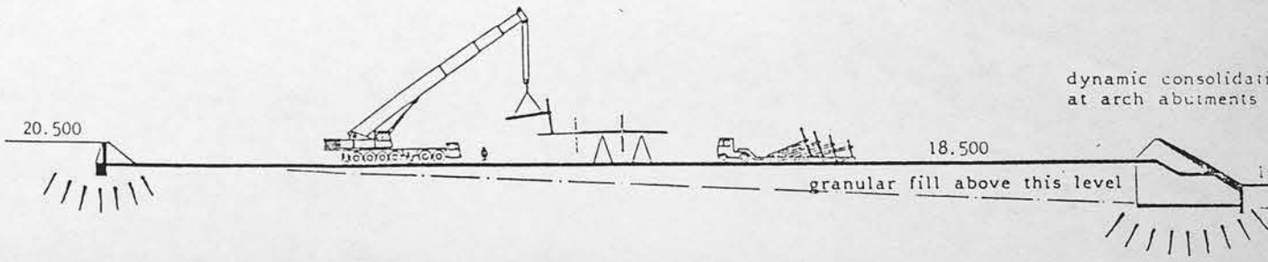
Gross internal floor area:	4,400 m^2
Area of mezzanine (optional):	660 m^2
Structural length of spine :	110 metres
Internal length:	71 metres
Structural width of building:	75 metres
Internal width:	69 metres
Maximum height of spine:	16.7 metres
Maximum headroom:	14 metres

(Marks / Barfield)

Assembly

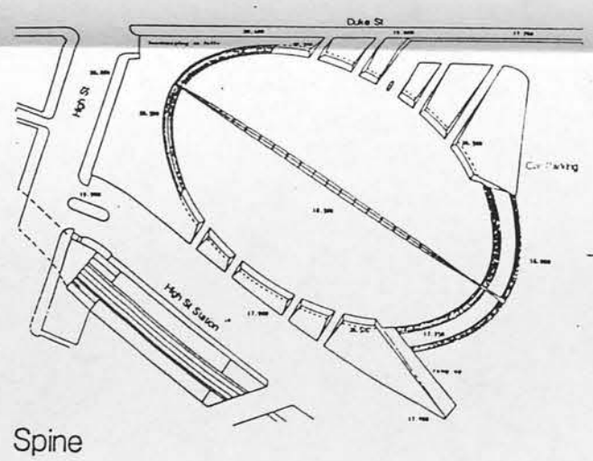


Toroid

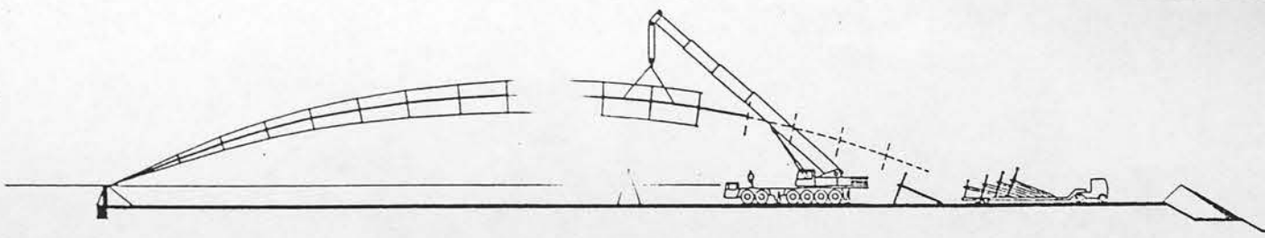


Stage 1

Stage 4

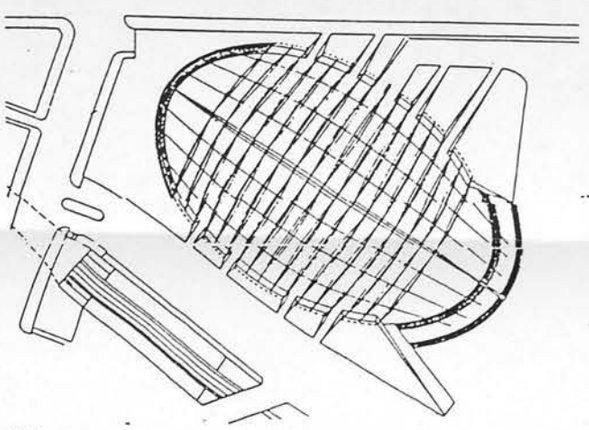


Spine

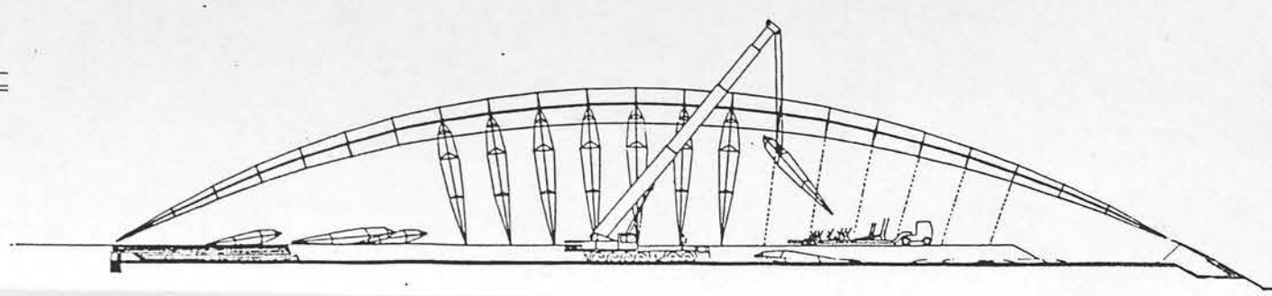


Stage 2

Stage 3

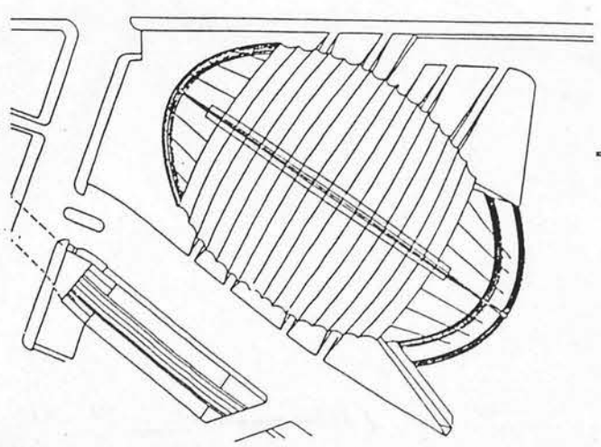


Ribs

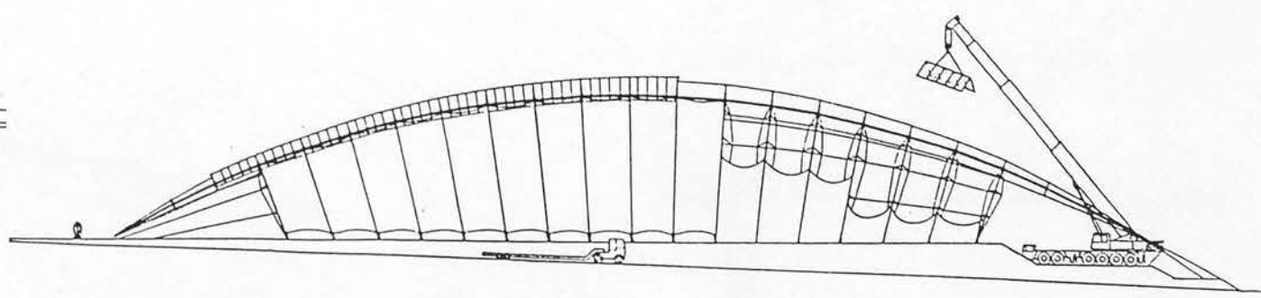


Stage 3

Stage 2



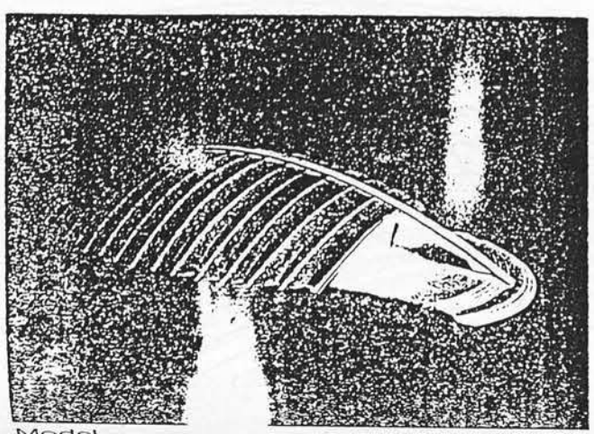
Skin



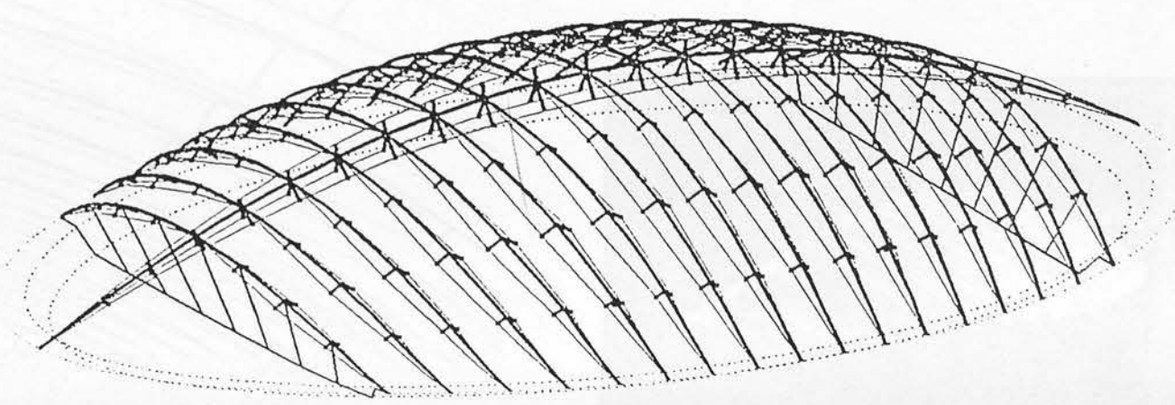
Stage 4

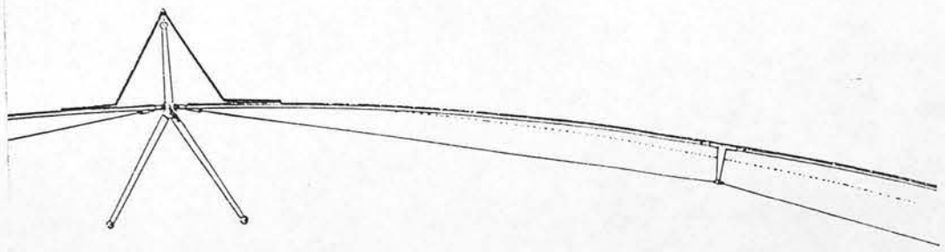
Stage 1

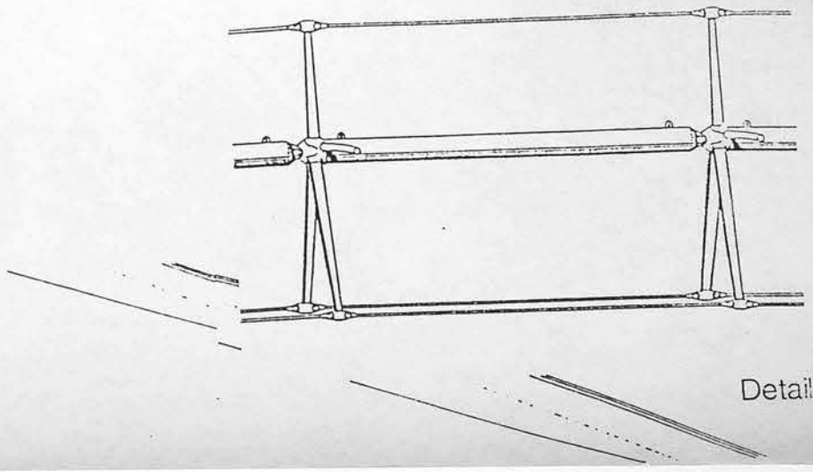
Dis-assembly



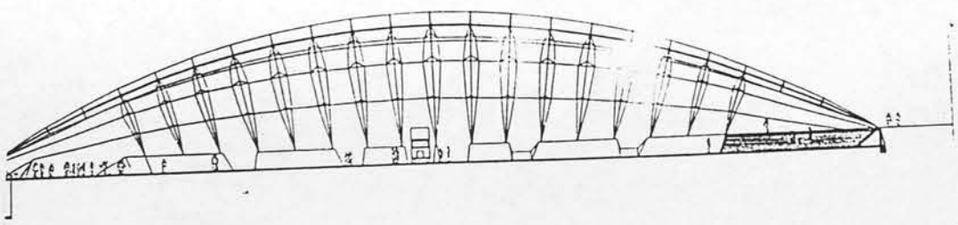
Model

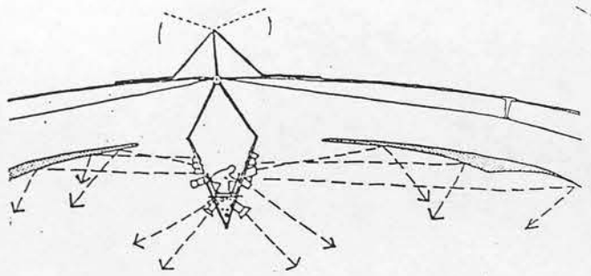






Detail





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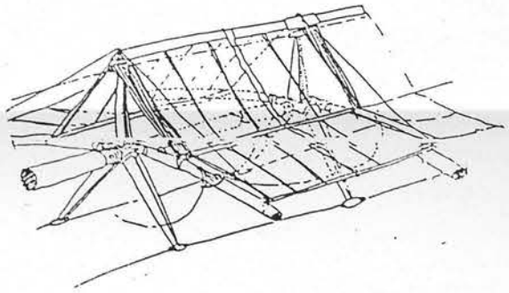

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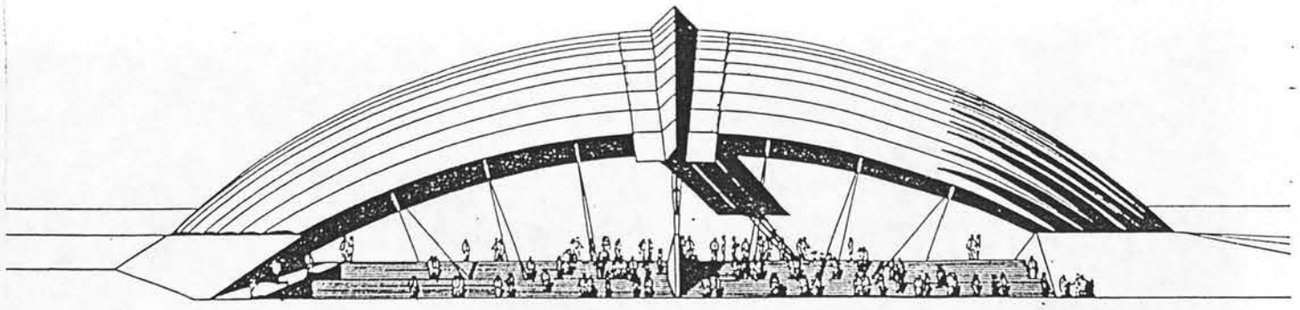
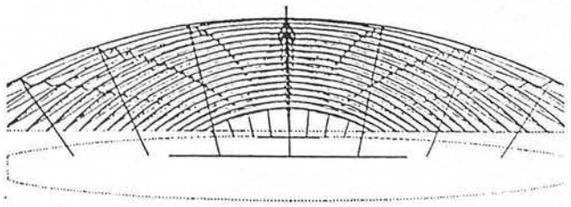
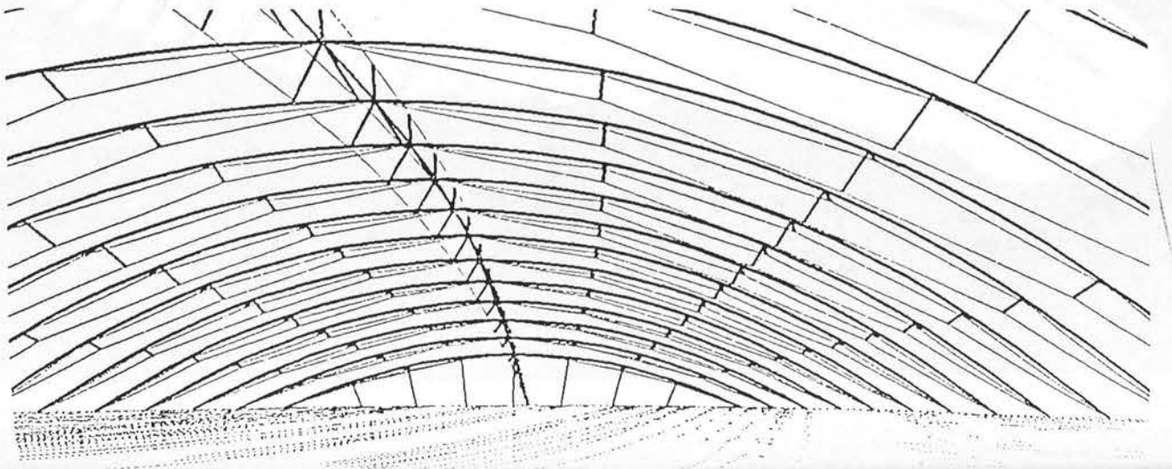
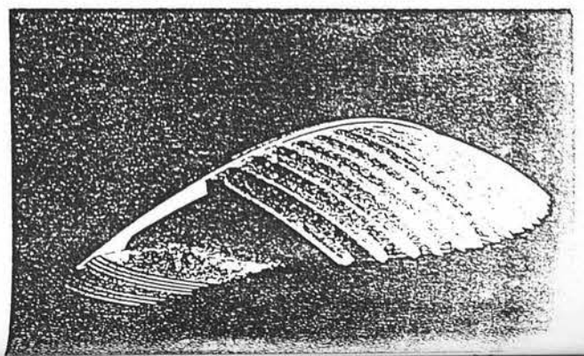
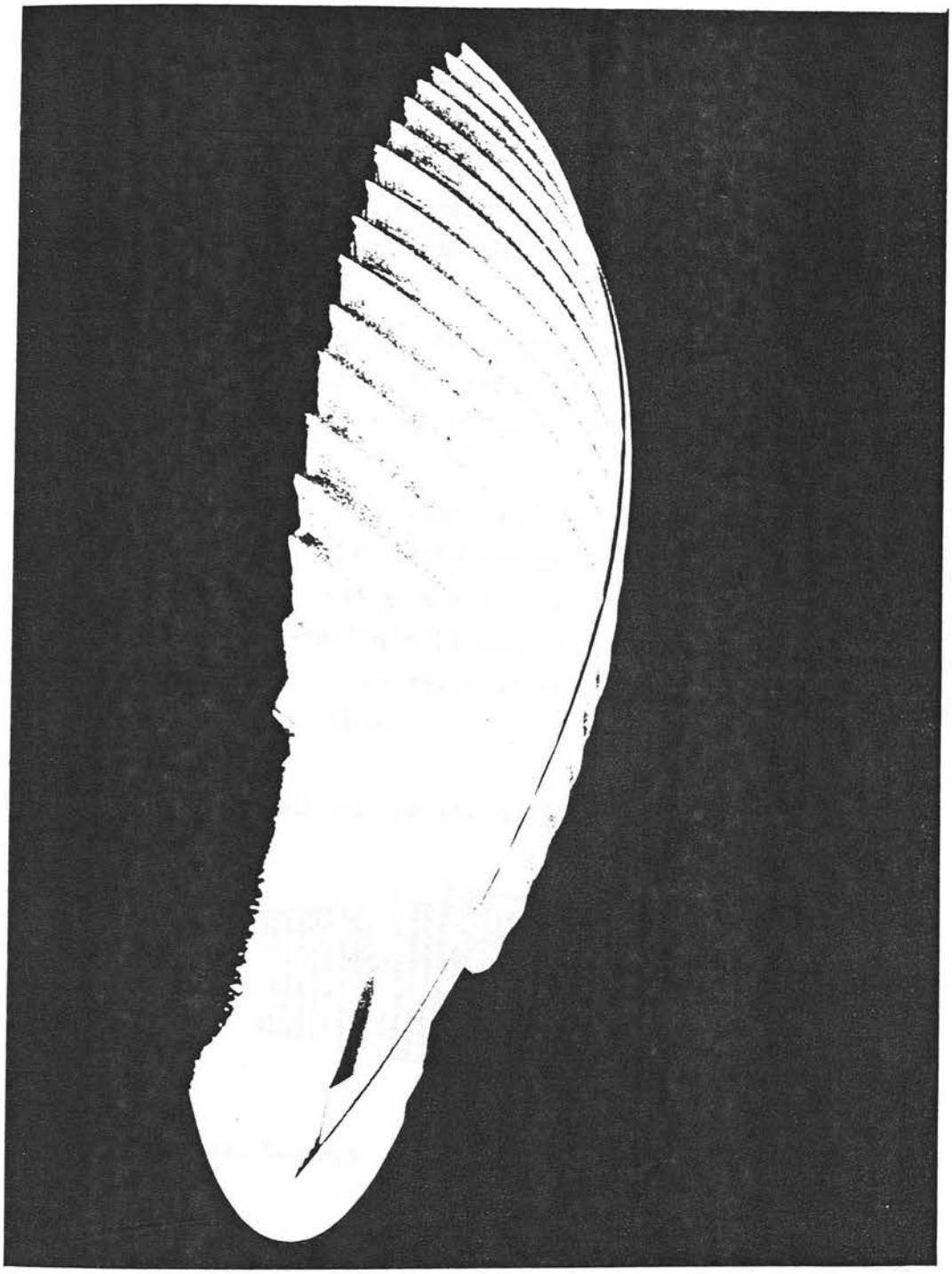










Entry by Chris McCarthy

THE GLASGOW EURODROME 1990

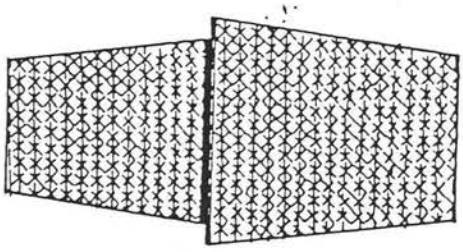
1.0 Introduction

The Glasgow Eurodrome will cater for a wide range of activities in 1990, Assemblies, Exhibitions and Performances.

The structure and fabric are so designed that they can be erected during 1989, completed by December and dismantled in tact for re-erection in 1991.

The structure has been designed and detailed in a way that will clearly illustrate the mechanics of the structure of the Eurodrome. The structure is a controlled mechanism as a crane or a weaving mill which will fascinate the spectator as the wheels turn in response to a load. A structure which in effective is alive.

The estimated cost of the 4000m² structure is £149 per m².



2.0 Structural Concept

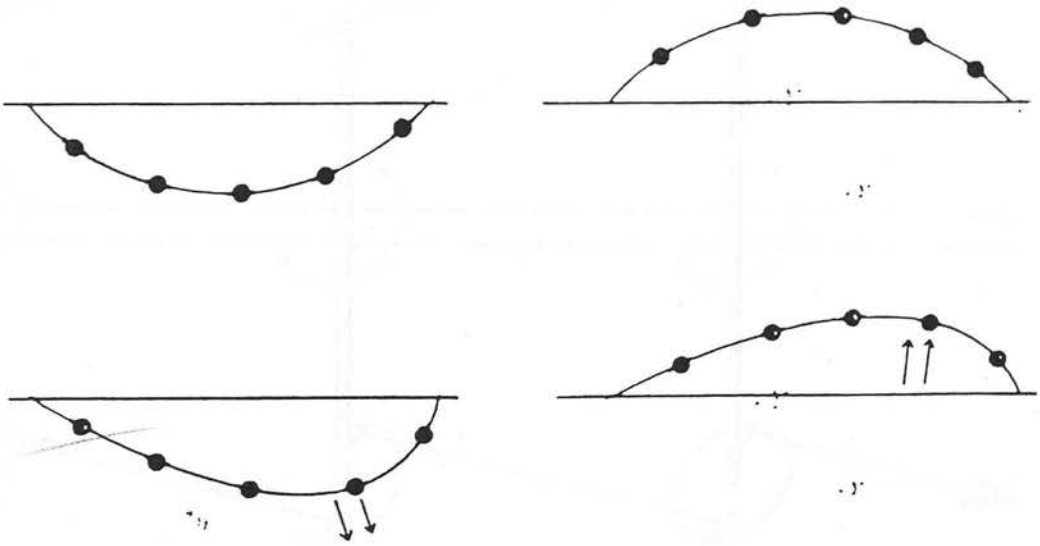
The function of the structure is to support the membrane which encloses the Eurodrome column free space. With the development of more flexible (high) performance fabrics, a less stiff, thus cheaper, structure is required to provide support. To optimise the structural principle of flexible structures a responsive structure has been developed.

A number of membrane/structure options were considered less stiff but conventional structures. However they did not optimise the flexibility properties of the membrane. Also it would be technically difficult to satisfy fatigue criteria. The notion of a controlled mechanism that physically

responded to applied wind and snow loads is a bolder more innovative concept for the designer and the audience. This structure would be educational in challenging the audience to understand the mechanics of the eurodrome. It would generate conversation and be a part of the exhibition or performance. " An Anamated Building!"

The hanging chain is the simplest responsive structure.

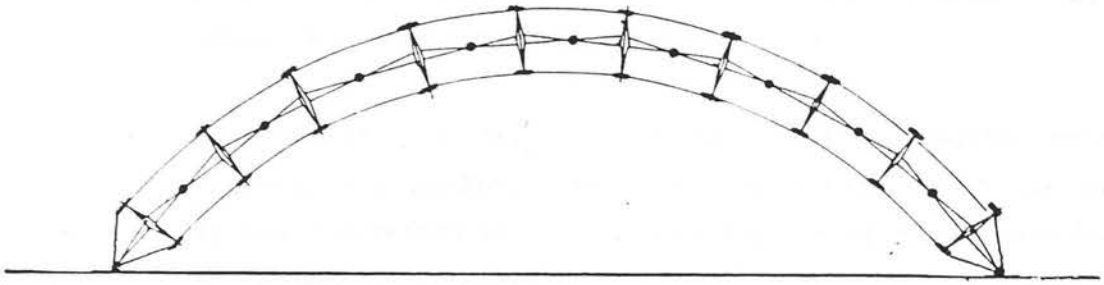
Inverted it is an arch. A principle which has been adopted since the Roman Engineers for form finding the most appropriate profile for arches.



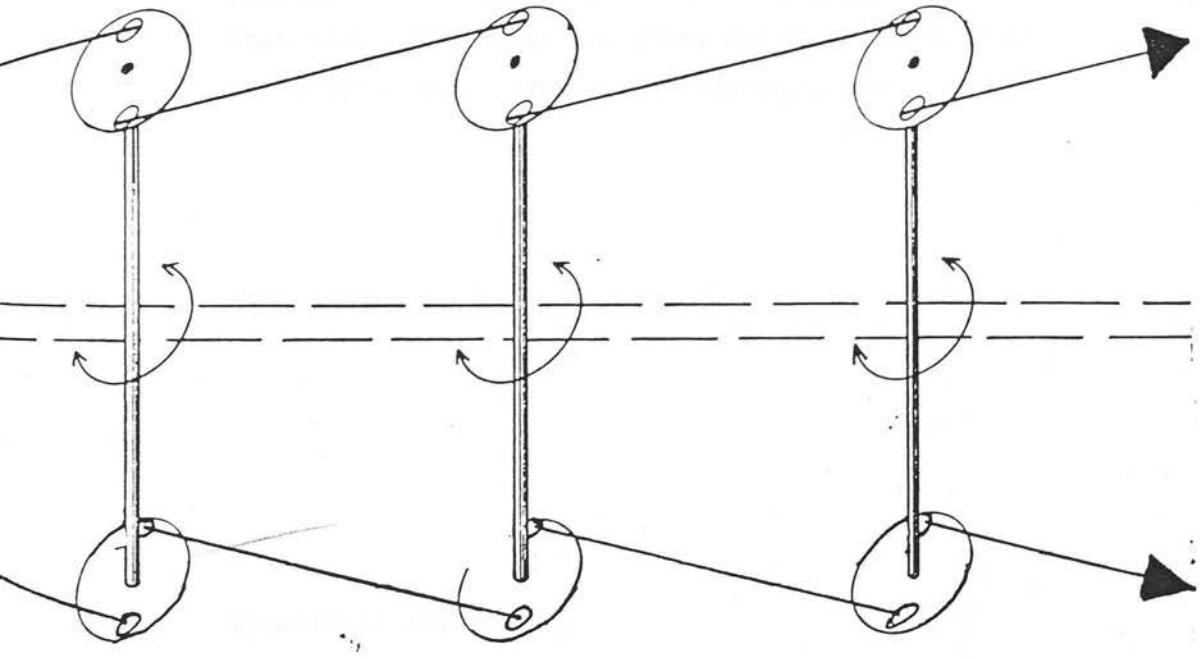
Under changing loads the chain responds to a more suitable shape (i.e. funicular shape). The amount of movement will depend on the magnitude and direction of the applied force as well as the self weight of the chain (c.f. design of suspension bridges).

The principles of the chain can be generated into a responsive arch.

3.0 Responsive Arch.



The Eurodrome arch consists of a series of compression star elements balanced by prestressed chains.



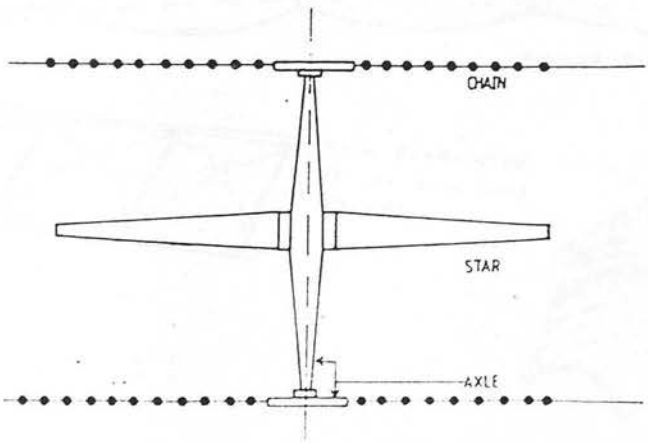
Linking the tension of the top chain to the tension of the bottom chain by the means of a vertical axle produces an interesting result. The tension in both the top chain and bottom chain will be maintained to be equal for all load cases.

The result is that the arch will respond to loading, (as a true hanging chain), until it has resumed its funicular shape again. The amount of movement will be controlled by the amount the chains are prestressed.

Considering the tension in the chains remains constant whatever the loading condition, the stiffness of the system is not determined by the Youngs Modulus of the chains but on the amount of prestress.

Stronger and stronger steels are forever becoming available, yet their elasticity remains unaltered. Thus to make maximum use of high strength steels, structural systems should pursue stiffness through the strength of the material rather than through the elasticity of the material. The use of high strengths steel ties are thus optimised with this responsive structures rather than if it was a stable one.

4.0 Structural Material



Star	-	Pressure injected magnesium castings
Axle	-	Engineering Plastics
Chain	-	High Strength Steel

5.0 Pressure Injected Magnesium Castings

The pressure injected magnesium castings:-

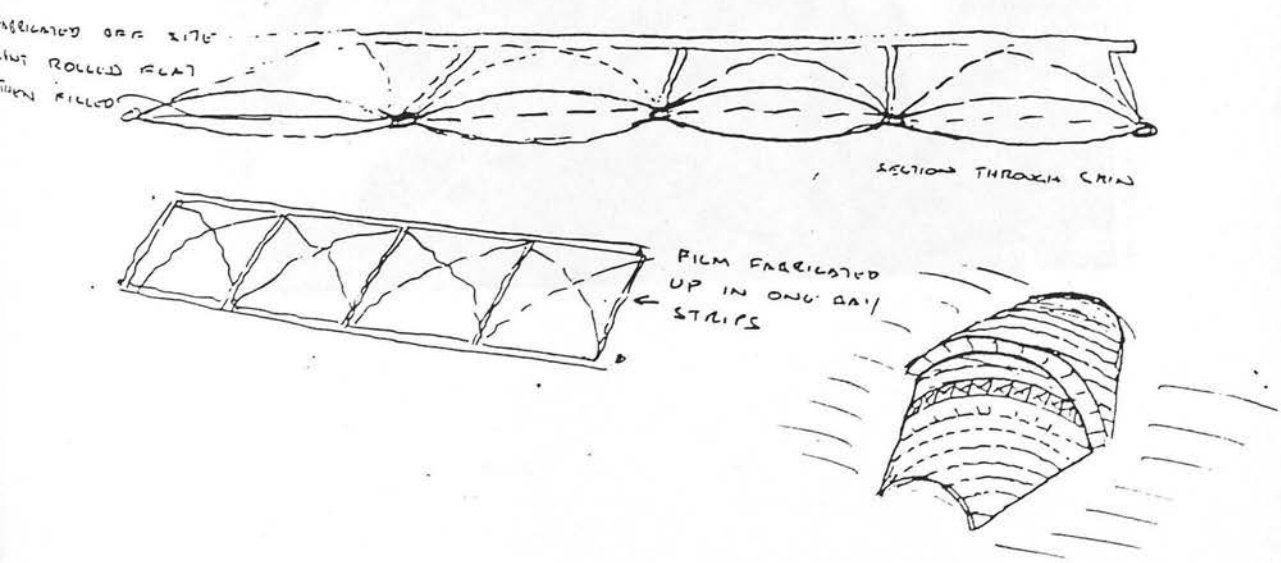
- are excellent economy for complex shapes with a great deal of detail
- can be mass produced
- are the ethic for a Glasgow tradition for cast iron engineering.

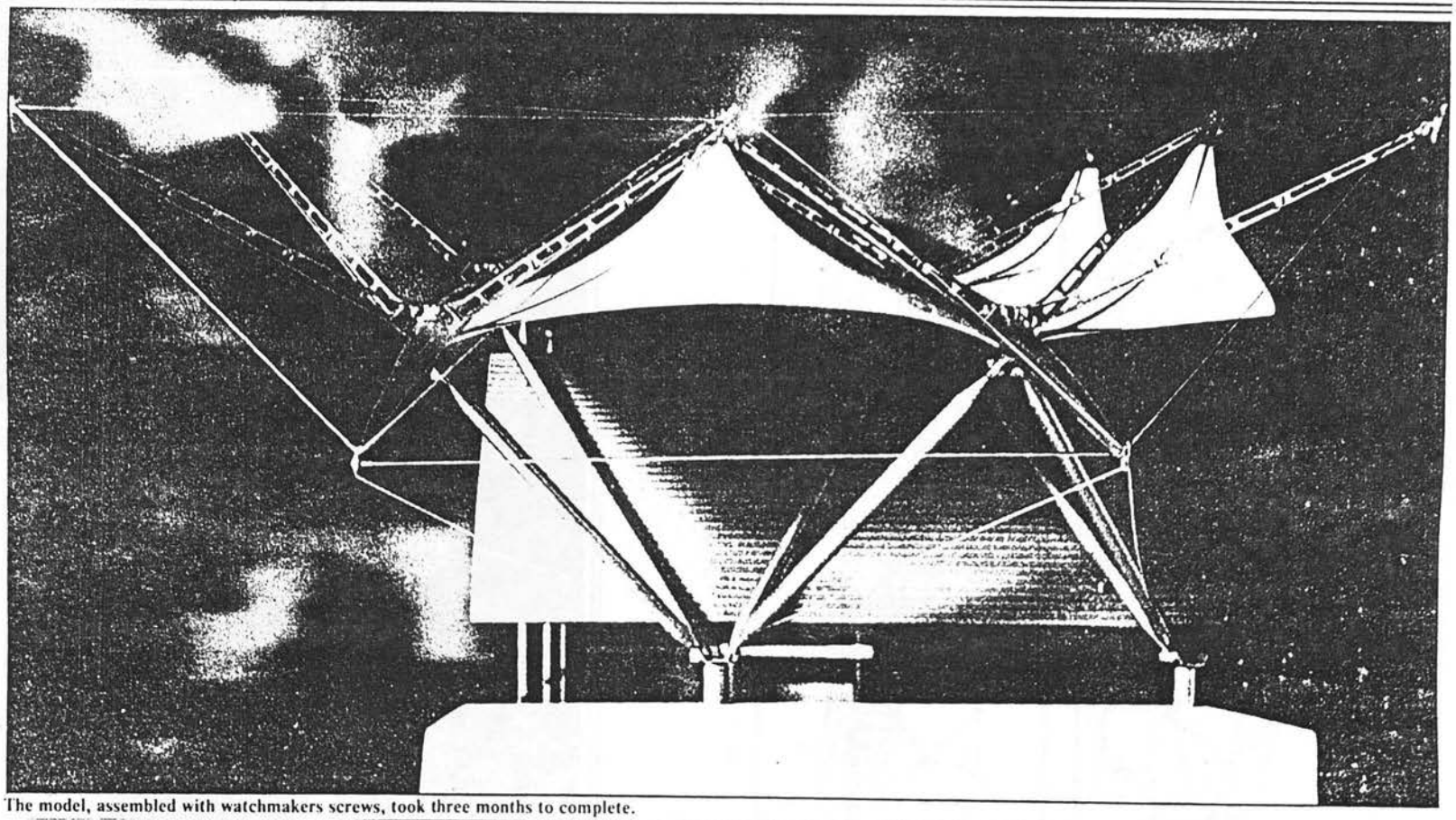
6.0 Engineering Plastics

Tufnol Laminate, with its high mechanical strength and excellent frictional properties, is ideal for the components of the axle. (Shaft bearings, seals and wheels). It is easily machined and it is available in rod, plate and tube.

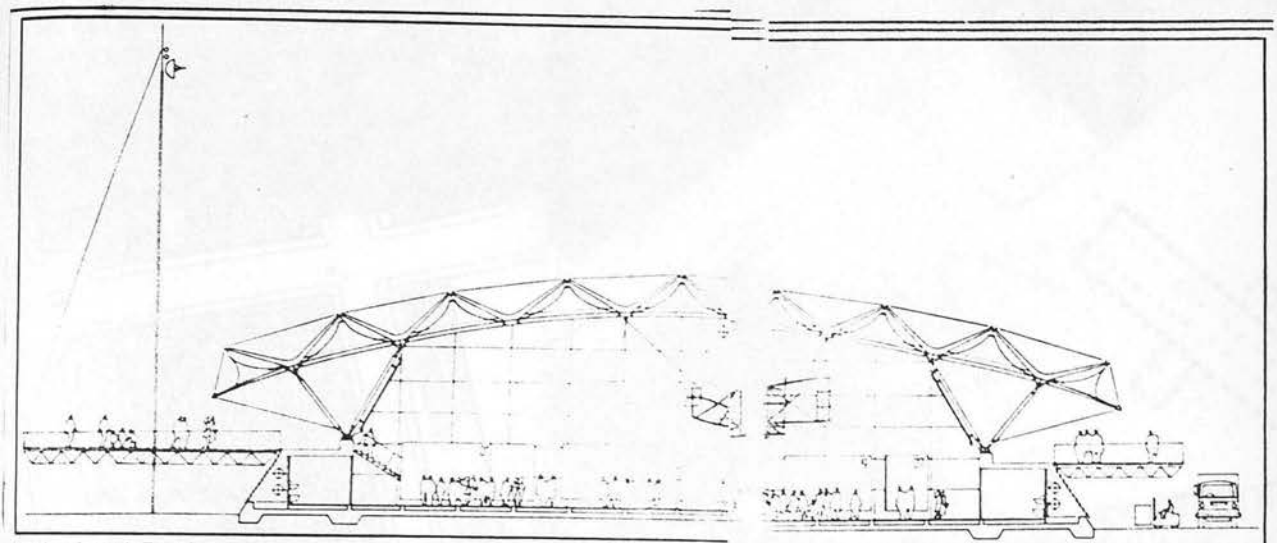
It is chemical and weather resistant. It is also a non-stick surface and is non-porous. It is a thermal insulator and will act as a thermal break.

7.0 Membrane Material

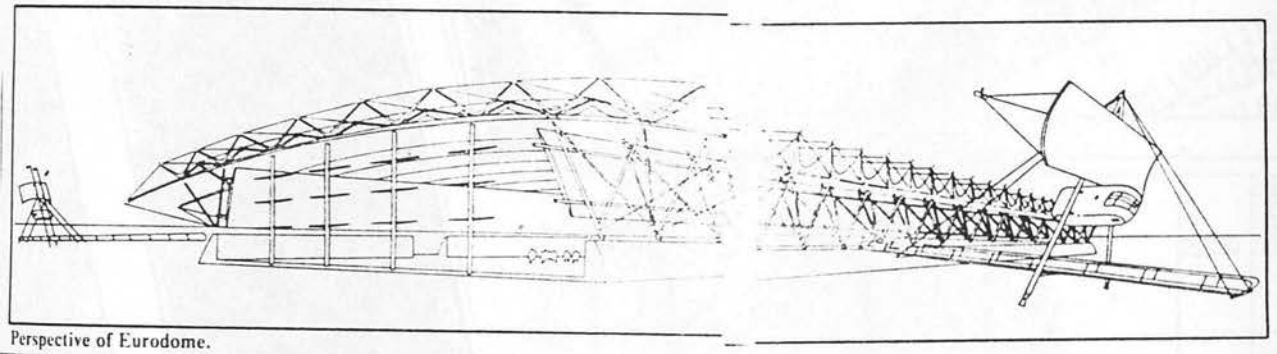




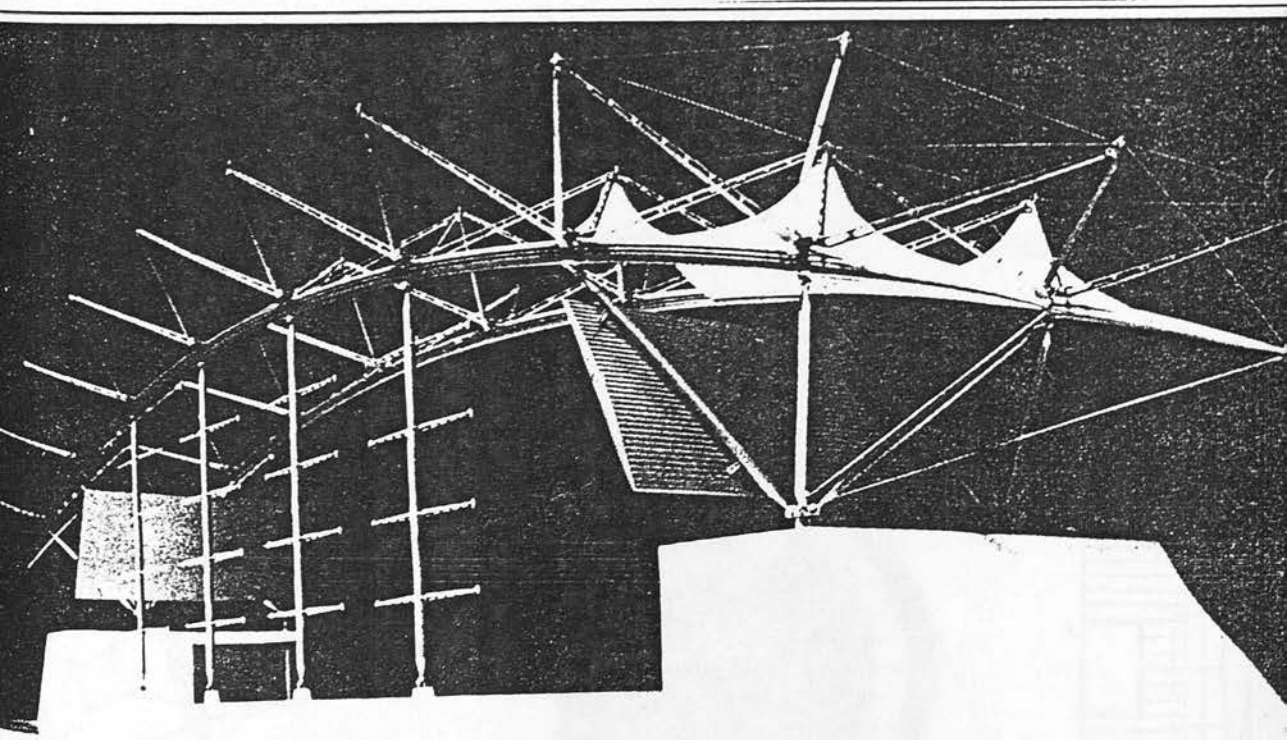
The model, assembled with watchmakers screws, took three months to complete.



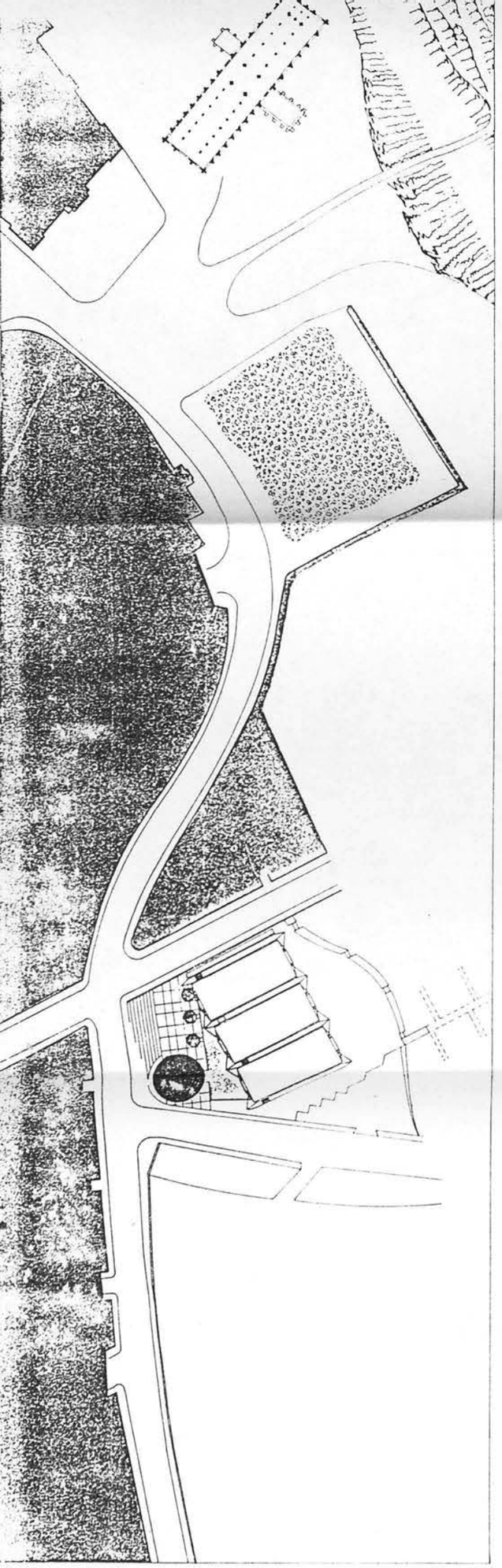
Section through Eurodome.



Perspective of Eurodome.

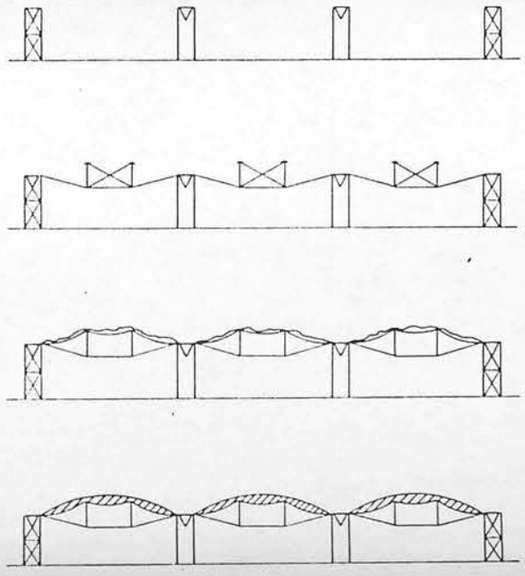


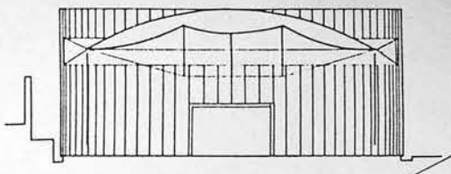
of the Glasgow Eurodome competition entry with Neil Thomas. Photos: John Donat.



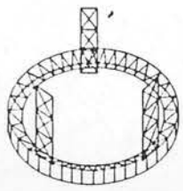
TE PLAN

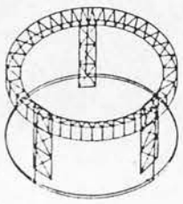
ERECTION

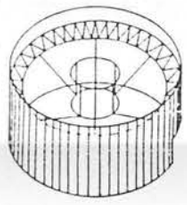


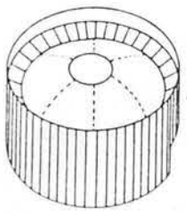


ARENA 1-500









The Concept

The building is a lightweight structure. Maximum snow loads are limited by heating the air, pressurising the space between the membranes, thus reducing the design snow load such that this is not the critical design criteria and the stress and deflection of the membrane and steel structure are limited.

In the event of a failure of the heating system, snow would accumulate on the structure with the risk of the inner membrane failing, but without the risk of the steel structure collapsing. (The safety factor is reduced to 1.1 under these conditions). In the assembly area, the inflated outer membrane pressurises the partially tensioned inner membrane, each panel of which is supported on 4 No. internal masts suspended below the membrane. This double layer achieves a moderate level of sound and heat insulation.

The risk of condensation may be avoided with this system alone, but would be subject to a more detailed study taking account of both the use of the space and performance of the system. In the event, it was proved to be insufficient. The risk may be eliminated by either heating the air pressuring the membrane or enclosing a layer of insulation between the membranes. The latter would also enhance the acoustic properties of the system.

The outer air supported membrane has the advantage of limiting the wind loads on the inner membrane, making the structural design of this element more precise and economic. The outer membrane would flex under very high wind loads. The design assumes it is inflated to a pressure of 0.5 kN/m².

The primary structure supporting the membrane is to be a triangulated truss with horizontal and vertical strength and stiffness. It carries the insulated metal gutters lined with a .....membrane and is stiffened where the membrane is attached to it by a half castellated section.

The trusses span 40m and are strutted horizontally at third points to resist the out of balance horizontal forces from the membrane tension and cable supports. The latter cables carry the masts propping the internal membrane.

The trusses are supported on steel columns with the columns on the north face being braced to take north south wind loads and the columns on the east and west ends being braced to take east west wind loads.

Foundations are to be reinforced concrete bases 1m deep, founded on the fill over the site following vibro compaction in these areas.

These foundations are sized for a snow load of 25 kN/m² and full wind loads to give a bearing stress of 40 kN/m². In the event of failure of the heating system, overstress of the foundations is possible, but unlikely to give rise to any distress in the structure.

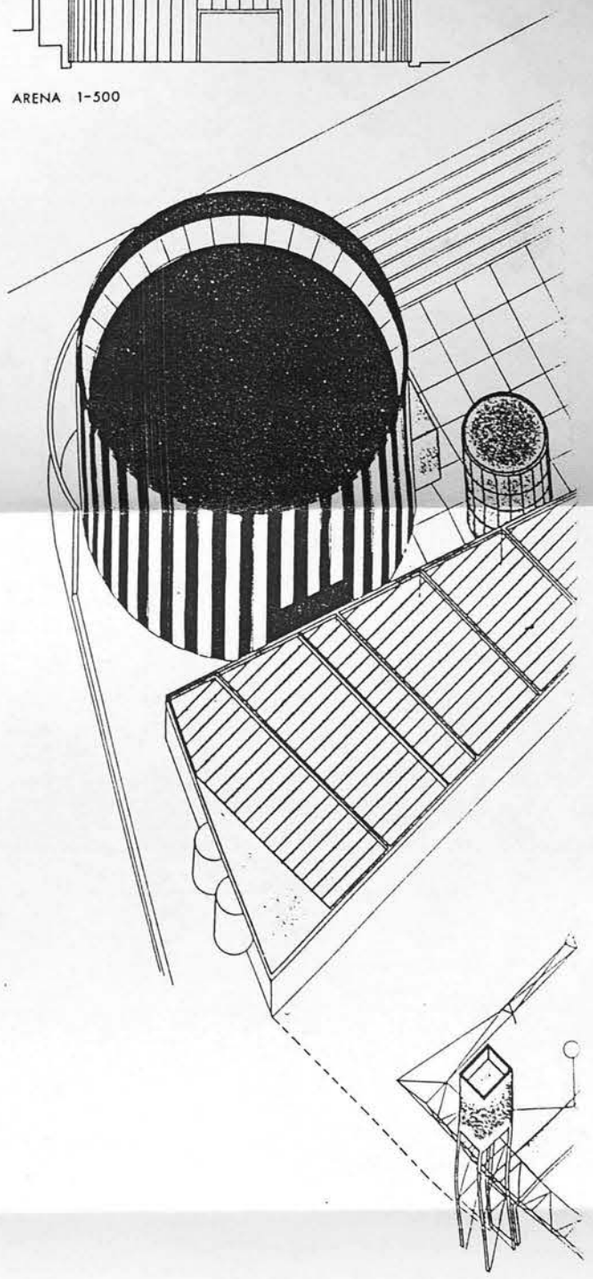
The fill on the site may give rise to some settlement during the life of the building, due to further consolidation of the soft clay layers. This would probably be proportional to the thickness of the fill and as the structure is articulated would not cause any significant distress in the frame.

The Reception

This building is a simple beam and metal deck structure, articulating the space between the arena and assembly. It has a proprietary flat roof and is envisaged to be specific to this site. It is low rise, founded on small mass concrete foundations on consolidated fill.

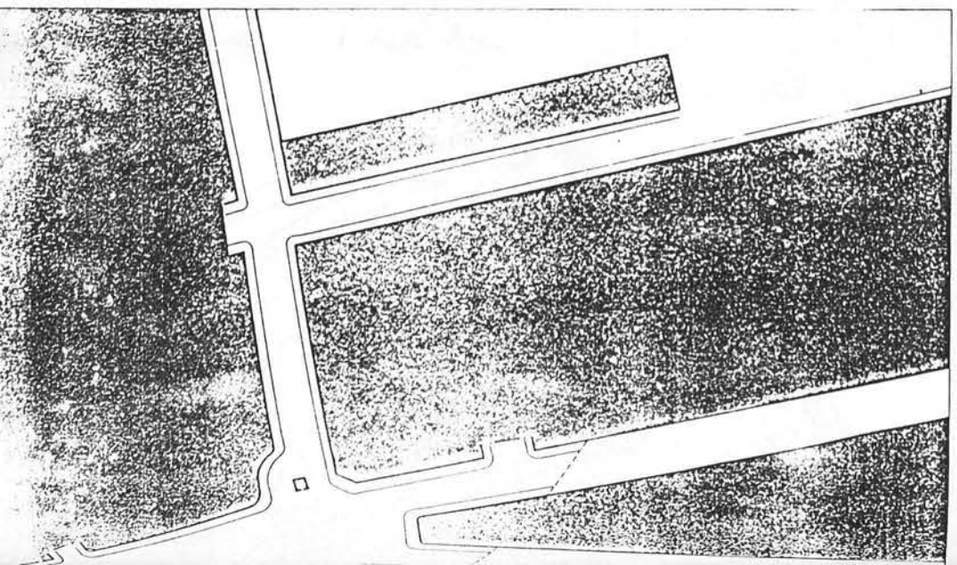
The Arena

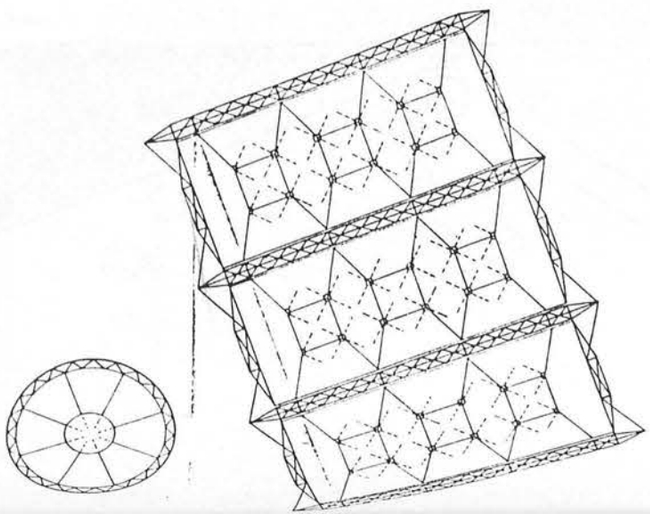
Whilst the arena may be a space required within the assembly building (the brief and answers are not clear), the opportunity of separating it from the latter space allows for the solution of the specific problem of limiting the noise of special events, such as rock concerts, to be made within cost limits.

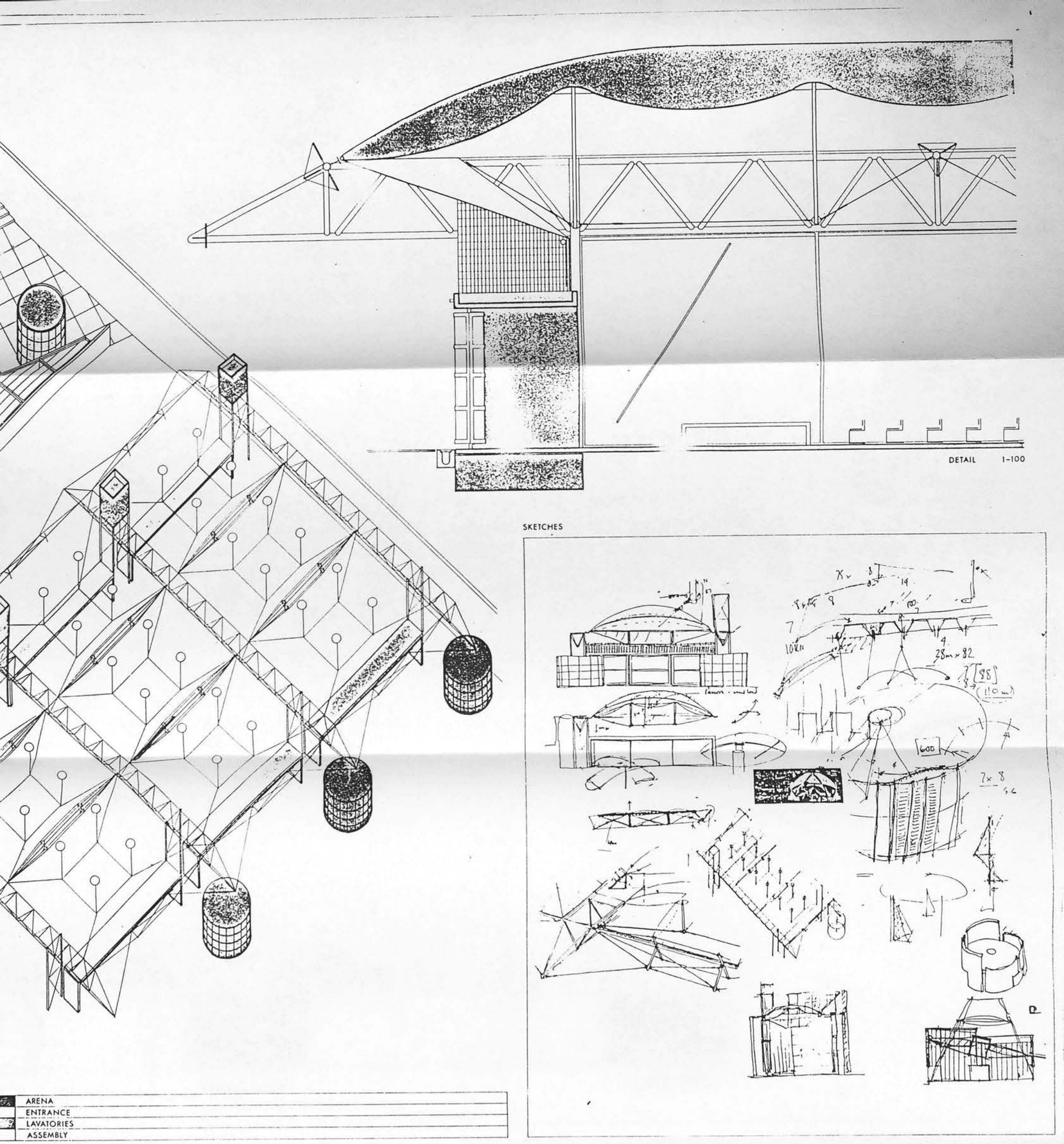


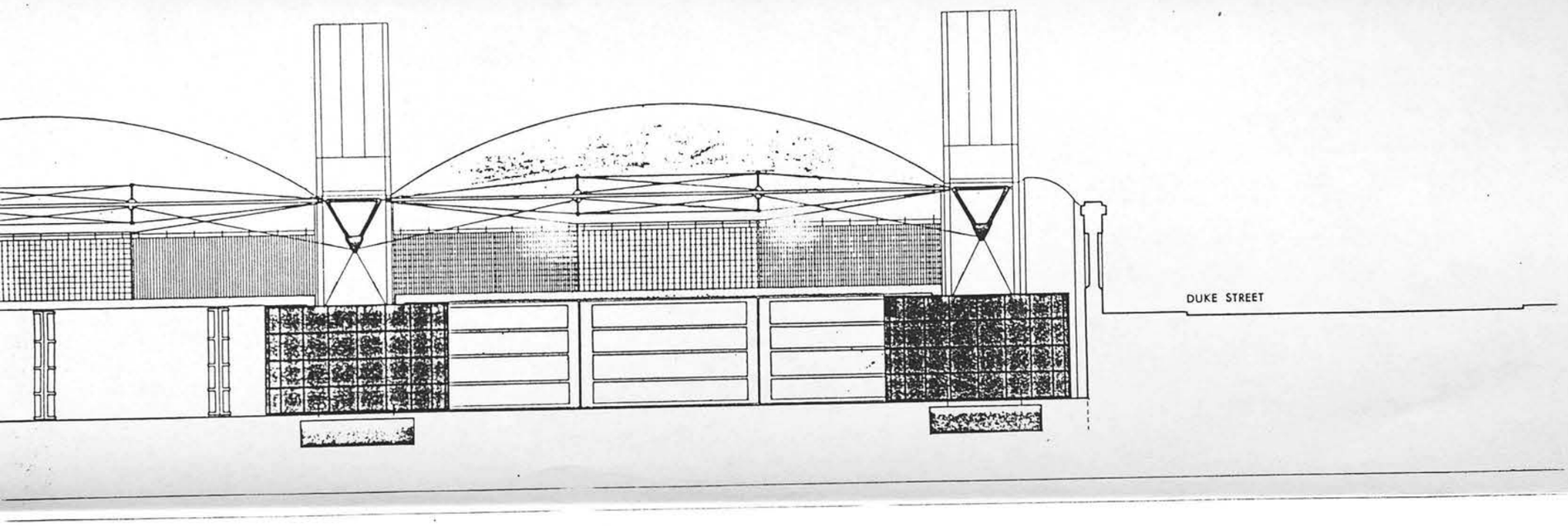
(Mark Whitby)

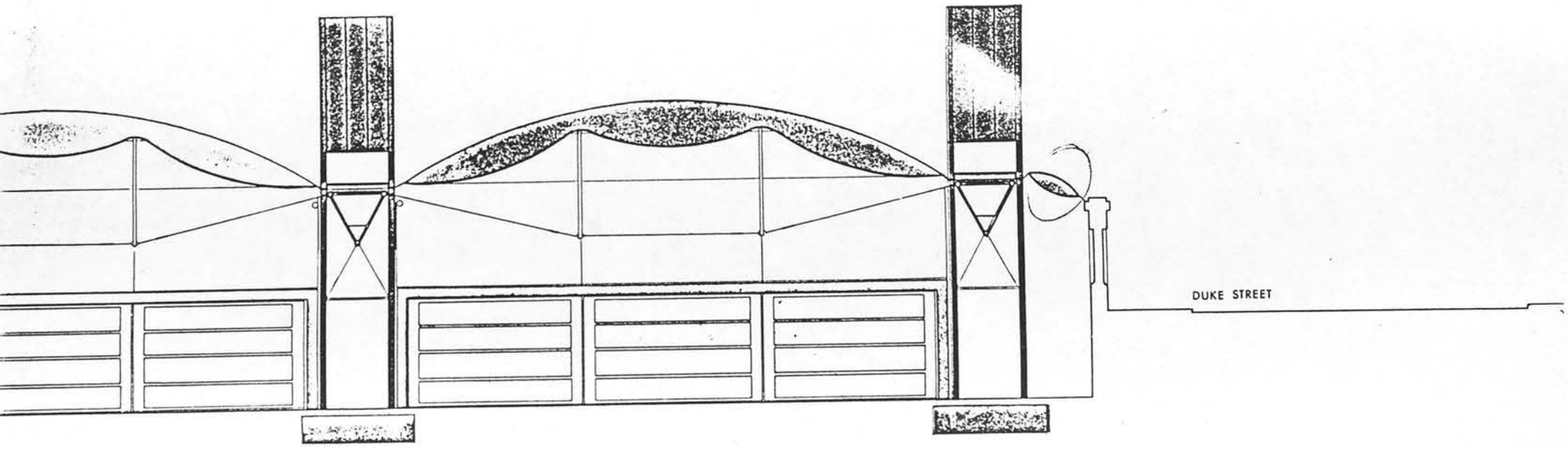
EFLECTED PLAN






DETAIL 1-100
SKETCHES
ARENA
ENTRANCE
LAVATORIES
ASSEMBLY
D PLAN
FOUNDATION PLAN
1-1000


DUKE STREET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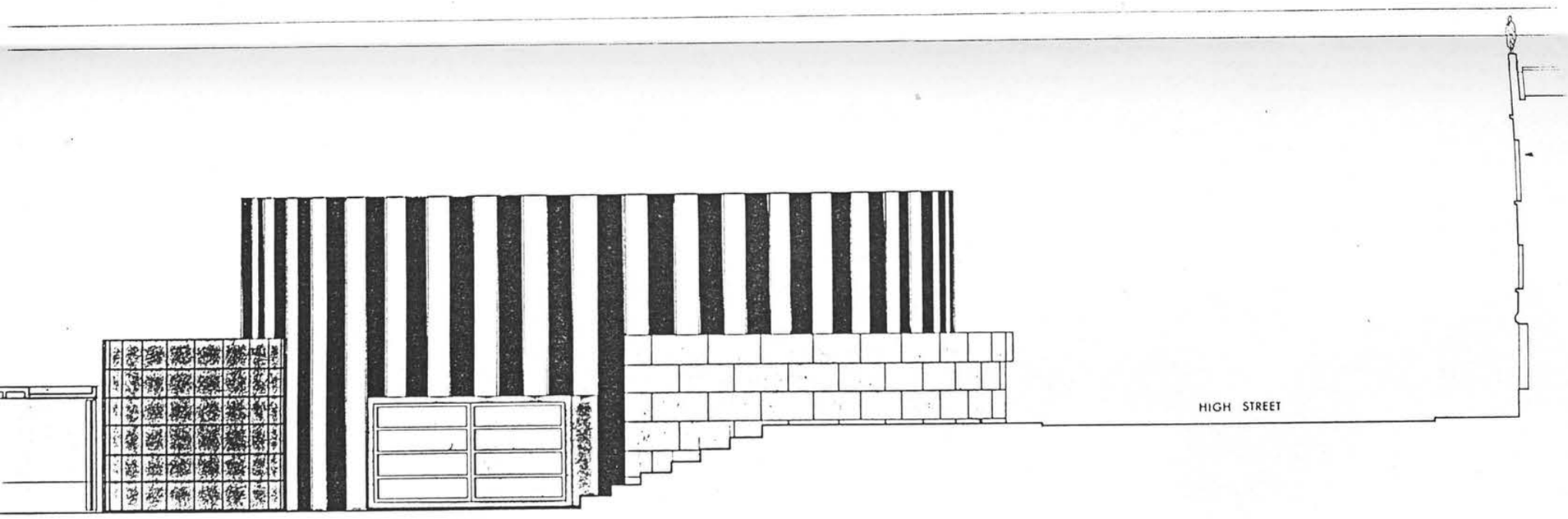
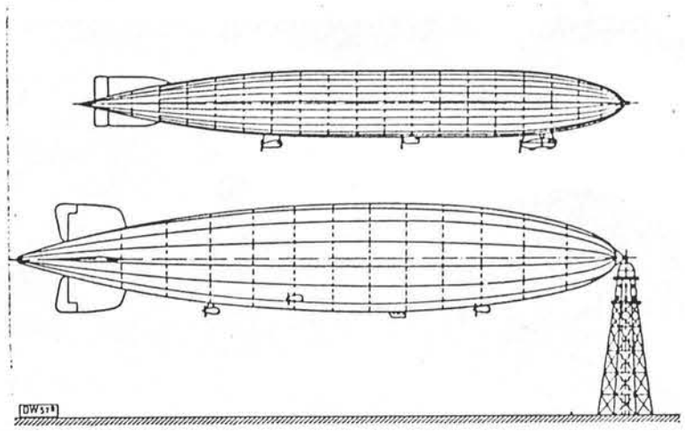
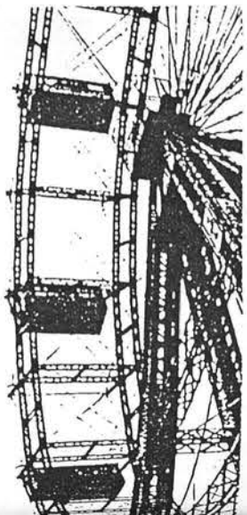
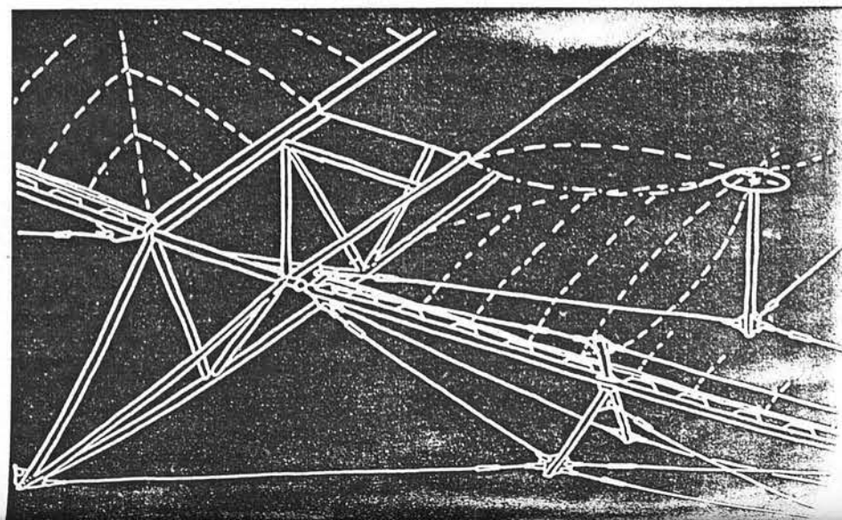
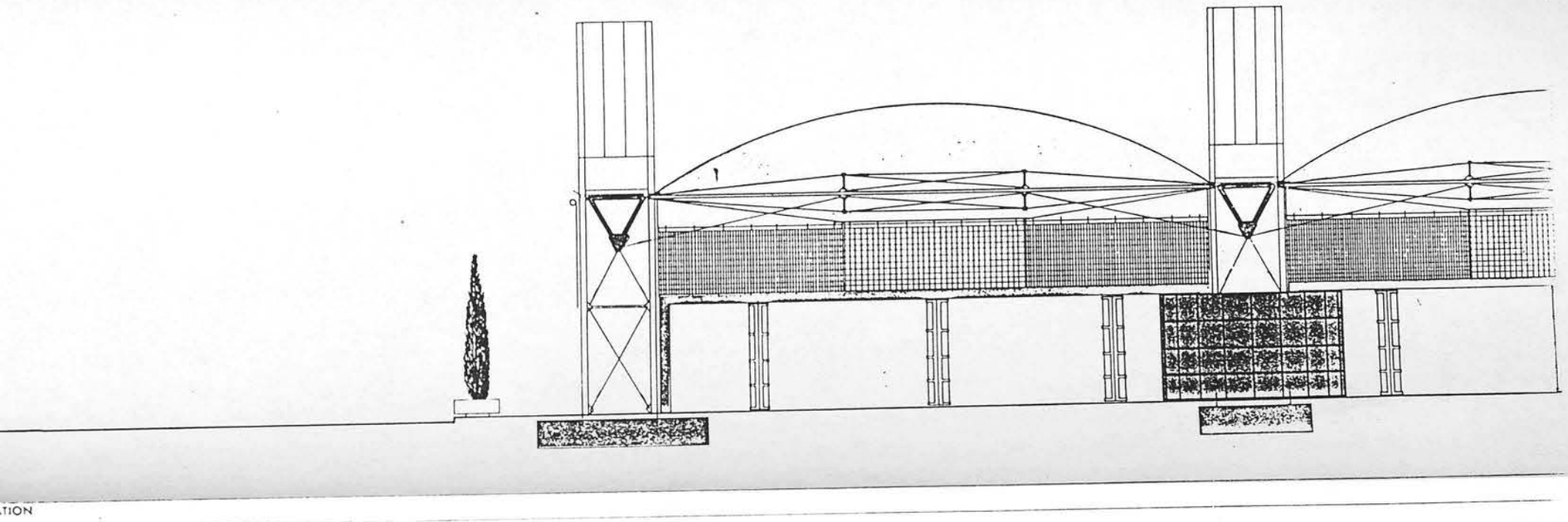

HIGH STREET









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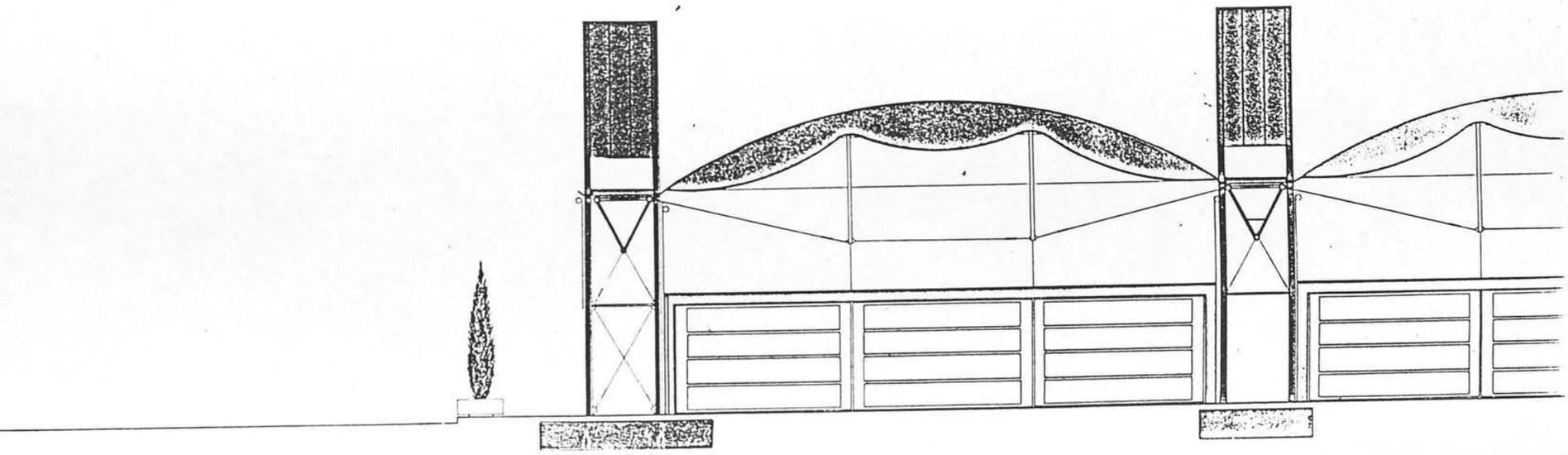
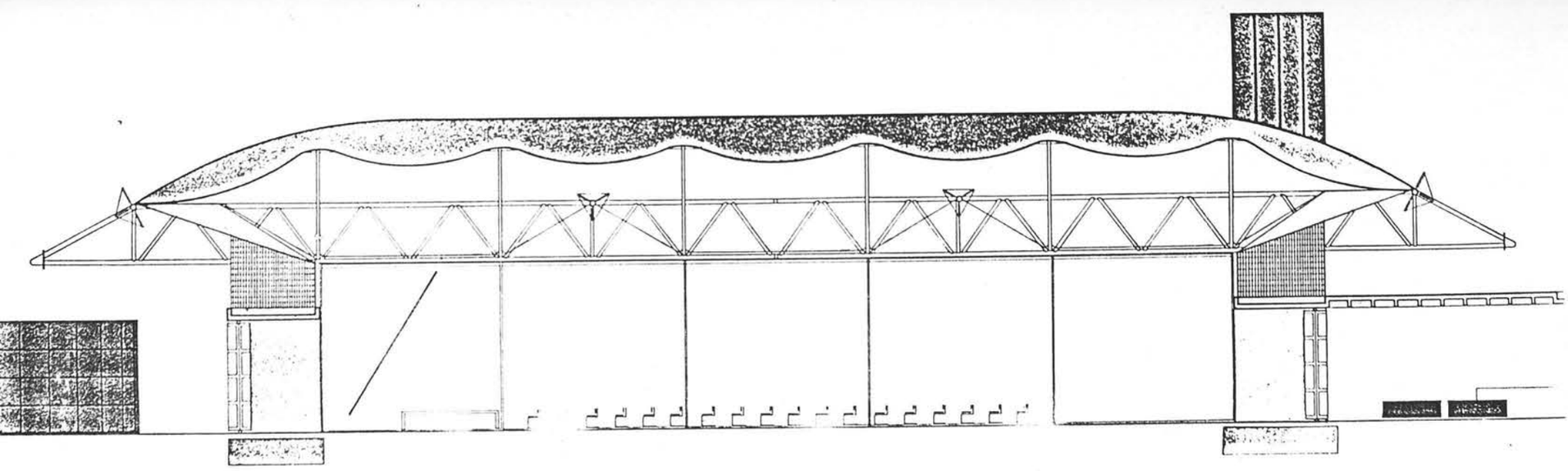
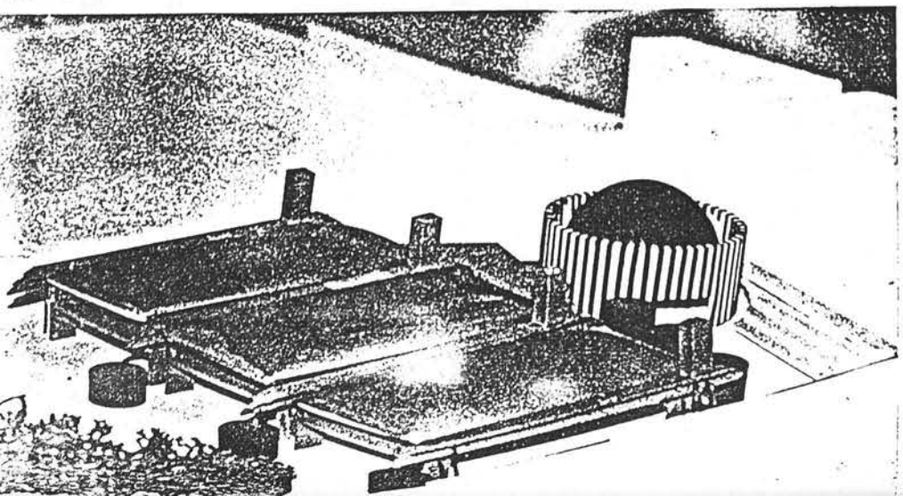
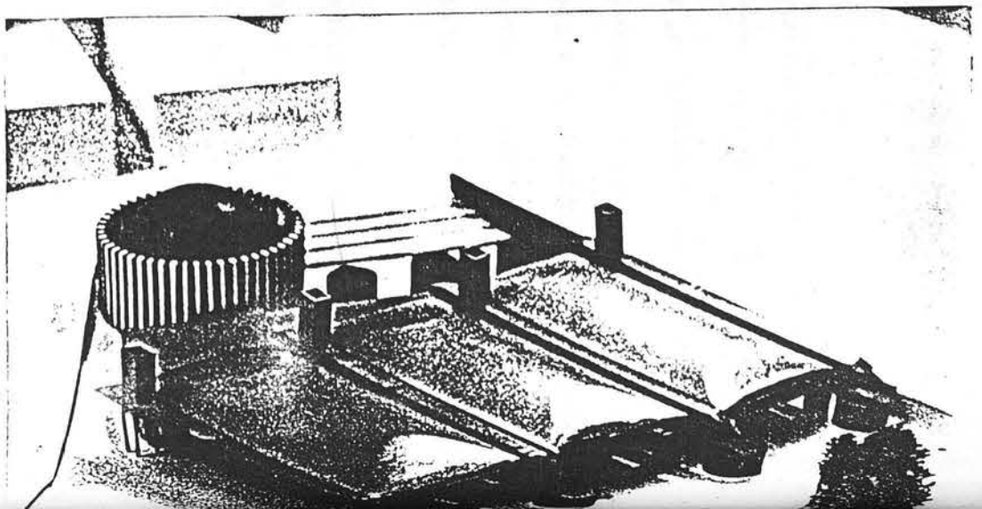


LOOKING WEST A



LOOKING SOUTH B





CASE STUDY 30: THE SELECTED WORKS OF SANTIAGO CALATRAVA

Calatrava's approach to Architecture and Engineering is primarily visual and sculptural.

> "The bringing up of a sense of beauty through the section seems to me very appropriate. In my work the structural, tectonic device is most important, thus, I insist so much on the section."

Foldability mobility, sculpture and the idea of the kinematic system applied to construction are of particular interest to Calatrava.

As in the case of the Swiss cement pavilion 1989, the construction was likened to a giant mobile sculpture.

"Crystallised movement (related) with... sculptural works of Rodin and Brancusi express mobility as an implicit concept of strength.

In this respect, Calatrava focuses on the use of concrete through which he introduces a new vocabulary of soft forms, of surrealist character, as a form of craftsmanship.

Zaera[1] likens Calatrava's aesthetic approach to the "einfuhlung" theory, a form of symbolic empathy enounced by Robert Vischer. The "Einfuhlung" proposed a perception of nature enlivened with human emotion. This theory recognised the value of an approach towards creating objects "impregnated" with emotion.

---

(1) Zaera A, The Living Structure, El Croquis, March 1989, pp 174 - 179.

This was an approach different from one which developed the objective quality of the product (as would the Rationalists), the concern was for the architects to give their buildings a life of their own, rather than expressing them through the plastic or symbolic qualities of the constructed form.

The approach involves aesthetic considerations which transcend the simple functional formal and constructive determination characteristic of the traditional architect's task and instead attempts to create order in inert materials such that each of its resultant parts would determine the original whole.

Zaera[1] wrote: "In architecture, the illusion of life is expressed through inflections and the different "stresses" of elements in "supporting, encumbering, lifting or suggesting".

The incorporation of the structural problem as a generator of order and the analogy between structures and natural organisms is a characteristic of "Einfuhlung" thought in relation to Calatrava's architecture. This approach is distinctly opposite to the historical abstraction of styles.

The forms created by Calatrava cannot be interpreted as abstract interpretations nor do they commence with the possibilities of using techniques or succumb to a system of industrialisation, but are instead shaped as unique structures without obvious precedents. "The form is converted to the purest expression of the organism without using established codes to provide meaning. Calatrava replaces the code with the analogy, the sign with the shape, rationality with

logic, and the procedure with the project"[2]

Calatrava expresses movement in his structures and configurates form to transmit forces in ways which do not refer to the modulated arrangements implied by those of structural models.

Zaera[1] observes that whereas in conventional structural arrangements, their form and mass[3] tend to accummulate in areas next to the supporting elements Calatrava"s schemes tend to concentrate the mass[3] of the structure at a point other than the support, subjecting it to a scattering of forces, usually torsional or flexural, and with are dispersed centrifugally in all direction..."

Rastorfer,[4] by contrast, observes that the structural principles behind Calatrava's designs are simple, leading to statically determinate configurations. As with Nervi's and Torroja's approaches, the forms are influenced by moment diagram studies in the initial design stages.

Torsion rings and folded girders characterize Calatrava's work. The forms for the canopy design in Lucerne and the superstructure for the Stadelhofen Train Station are a result of designing for torsion in the structures.

"Unlike the simple linearity of tension and compression, torsion is a spiraling stress. Calatrava knowingly lifts the locus of the torsion away from the boundary of the struccture. The result is a dynamic

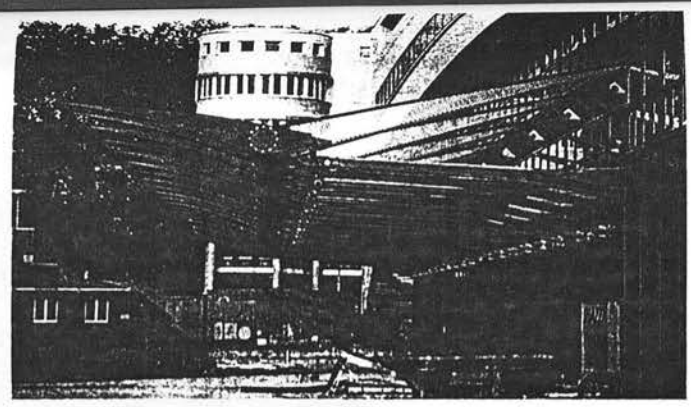
(2) Worringer, W "Abstraktion und Einfuhlung".

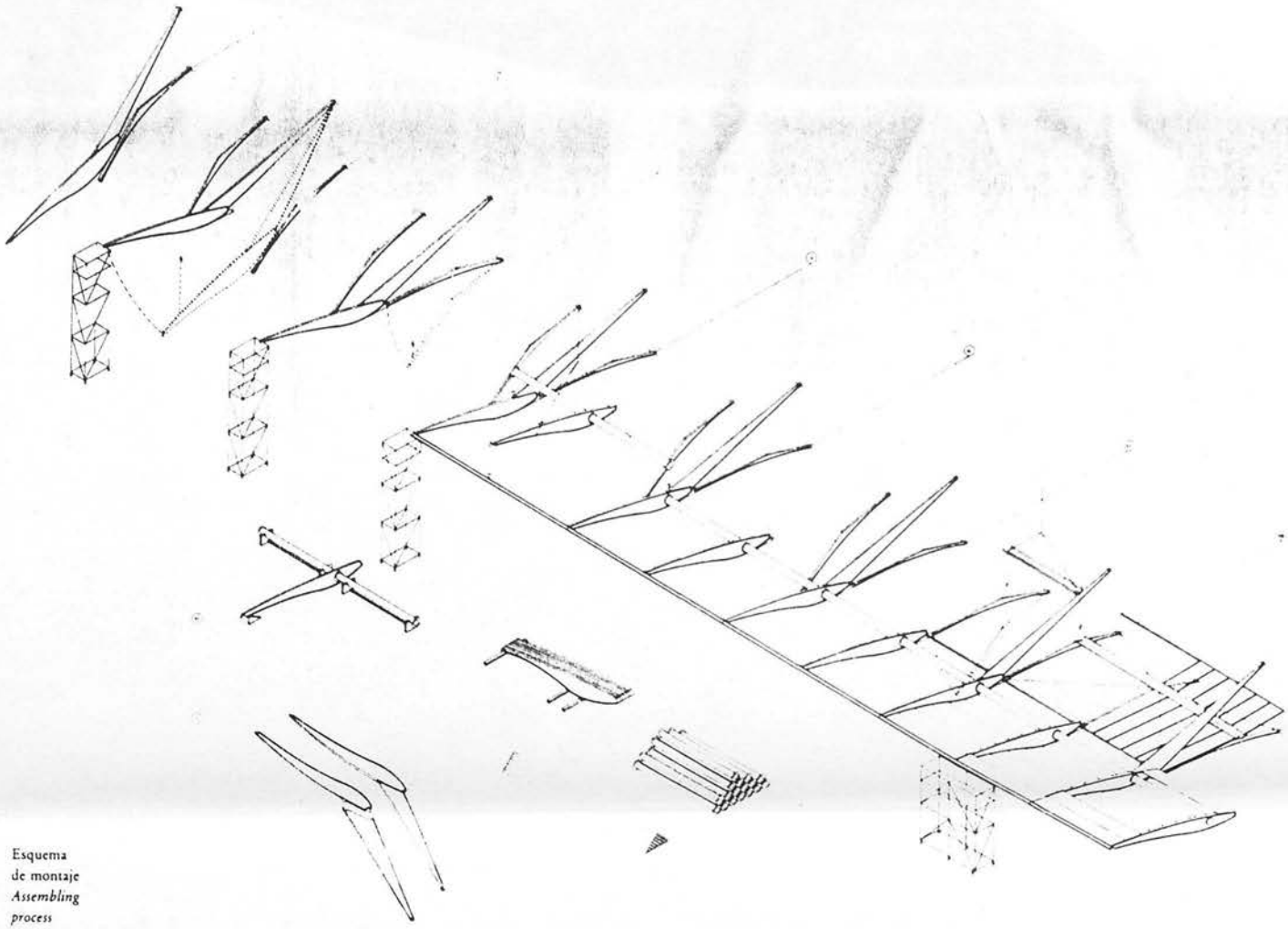
(3) Visual mass.

(4) Rastorfer, D, "The Structural Art of Santiago Calatrava", El Croquis, March 1989, pp 186 - 188.

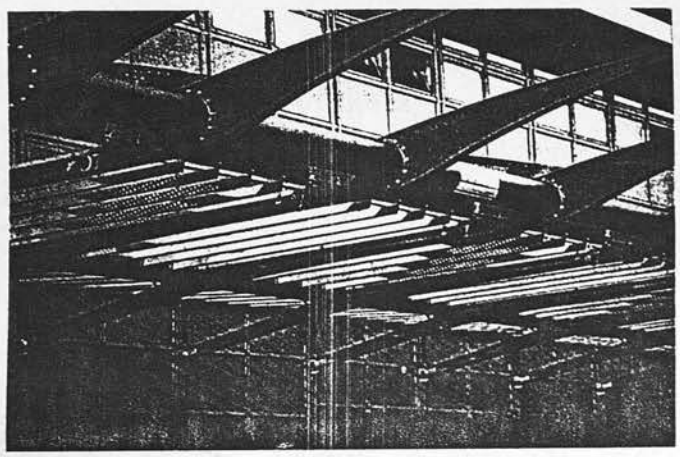
En esta página:
Vista general y detalles de la estructura
On this page:
Overall view and details of the structure
En la página de la derecha:
Dos vistas de la marquesina terminada
On the right page:
Two views of the completed canopy

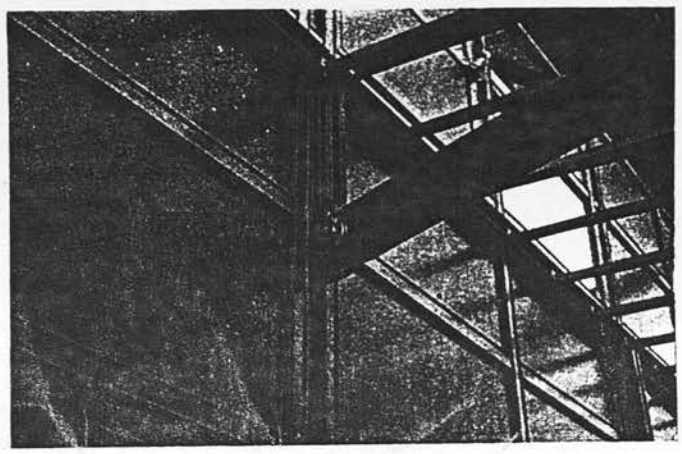
LUCERN POST OFFICE

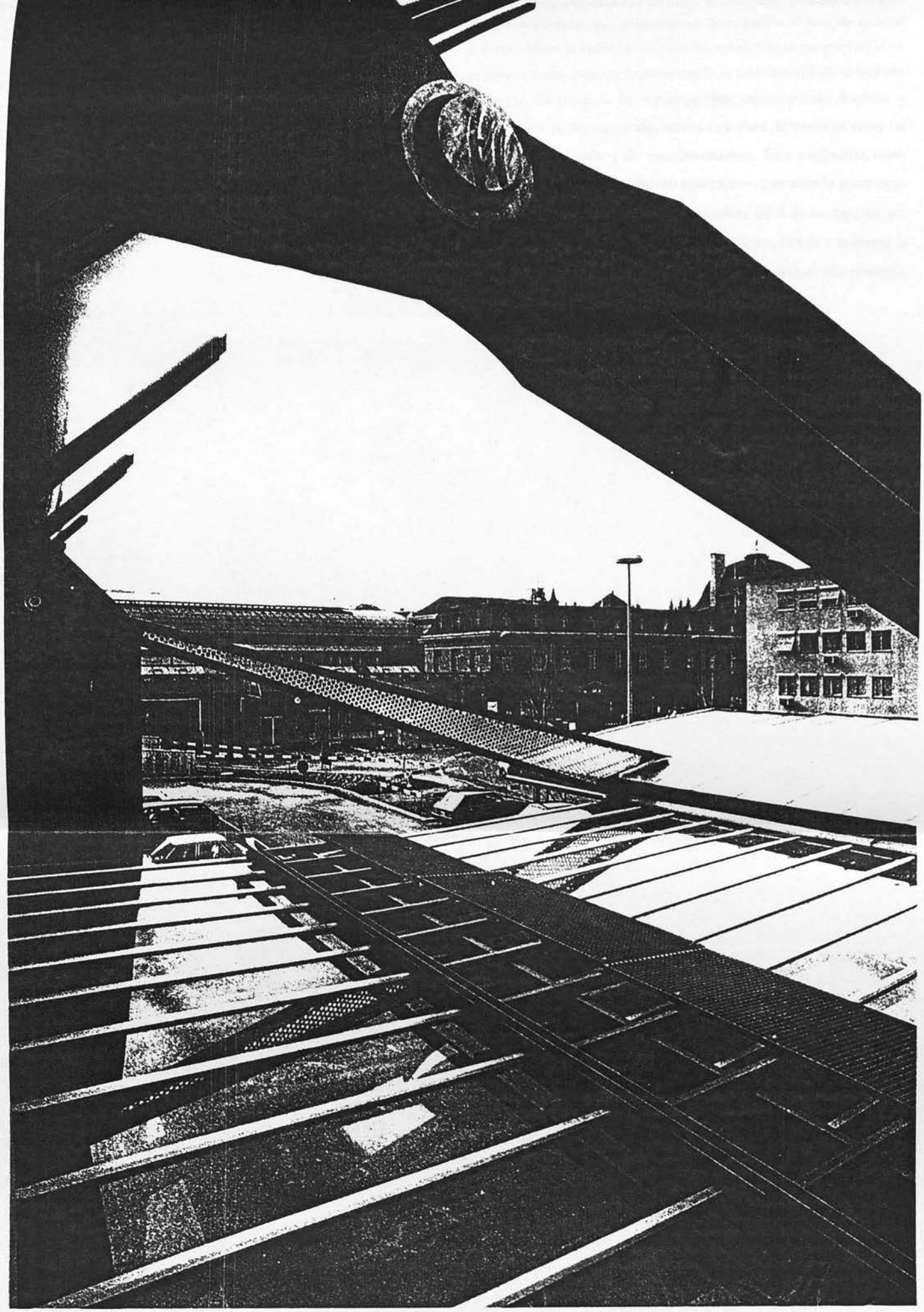




Esquema
de montaje
Assembling
process







Detalle de sección transversal

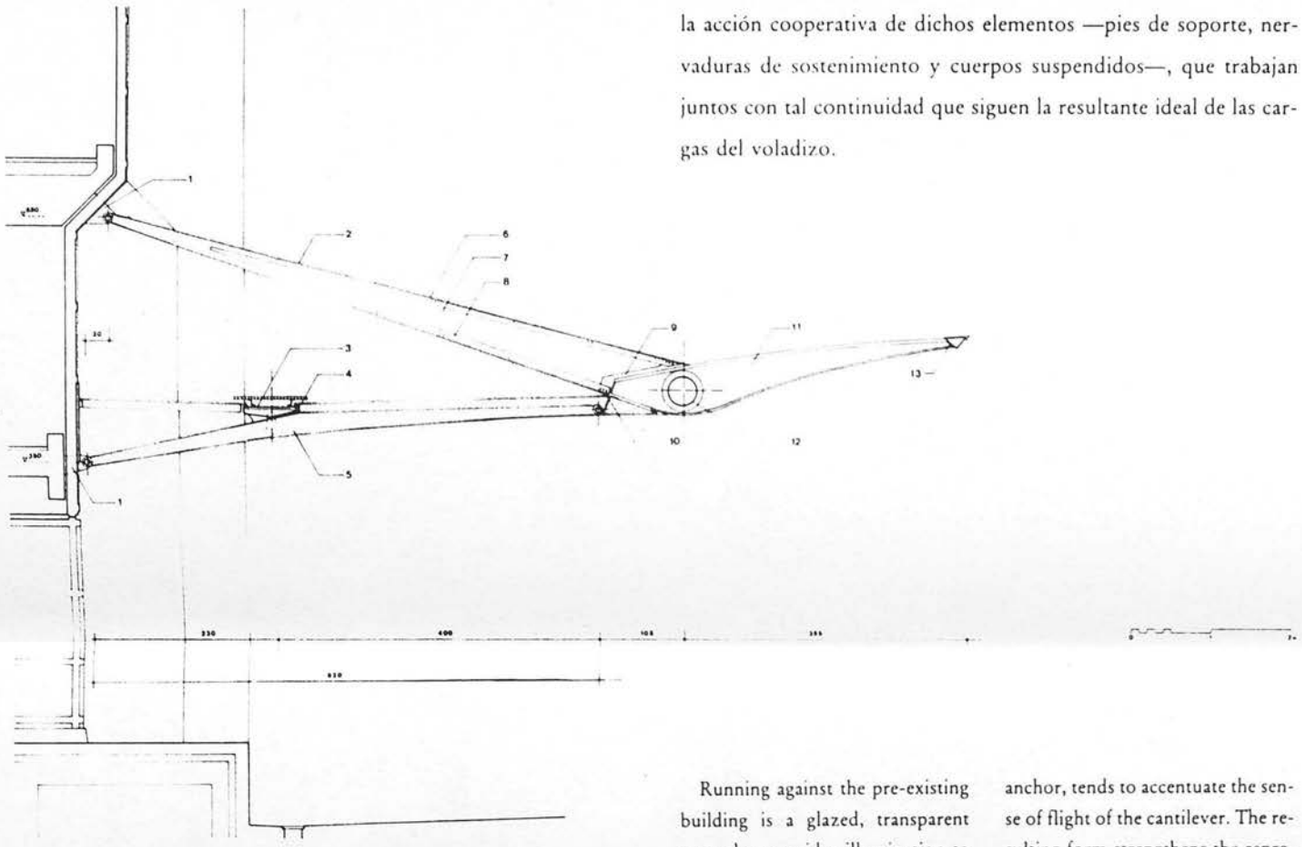
1 Placa de apoyo.
2 Tirante 12 unidades.
3 Rigidizador transversal del canalón.
4 Canalón.
5 Soporte 23 unidades.
6 Tirante superior.
7 Chapa intermedia.
8 Tirante inferior.
9 Pletinas de unión.
10 Angular de cierre.
11 Chapa de rigidización.
12 Tubo de torsión.
13 Angular de borde.

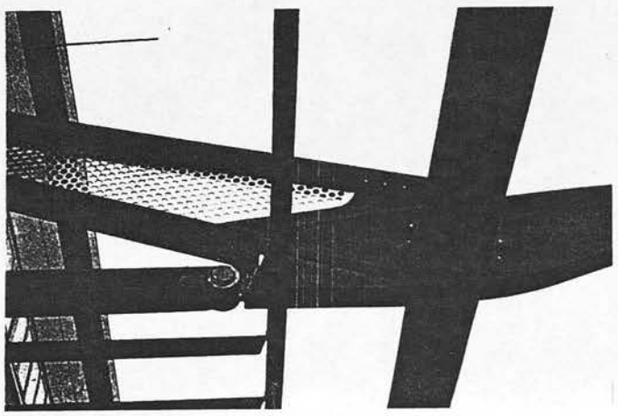
Detail of the cross section

1 Supporting plate.
2 Brace 12 units.
3 Transverse stiffener of the gutter.
4 Gutter.
5 Support 25 units.
6 Upper brace.
7 Intermediate plate.
8 Lower brace.
9 Joining plates.
10 Closing corner.
11 Stiffening plate.
12 Torsion tube.
13 Edging corner.

En la zona adyacente al edificio, se sitúa una estructura acristalada transparente, que proporciona iluminación al área de trabajo y vistas desde la fachada del edificio, revelando la geometría de su propio sistema estático y preservando la continuidad de la fachada existente. El voladizo de la marquesina, recubierto de aluminio y con sección en forma de ala, marca una clara delineación entre las zonas de circulación y de estacionamiento. Esta enigmática composición —resuelta sin esfuerzo aparente—, que sitúa la parte opaca, y visualmente pesada, de la marquesina lejos de su soporte, sobre el que se adosa la parte visualmente ligera, tiende a acentuar la sensación de vuelo de la estructura. La forma proyectada pretende reforzar el carácter representativo del edificio postal.

La eficacia de este sistema estructural, formado mediante la adición de elementos planos triarticulados, sólo es posible a través de la acción cooperativa de dichos elementos —pies de soporte, nervaduras de sostenimiento y cuerpos suspendidos—, que trabajan juntos con tal continuidad que siguen la resultante ideal de las cargas del voladizo.

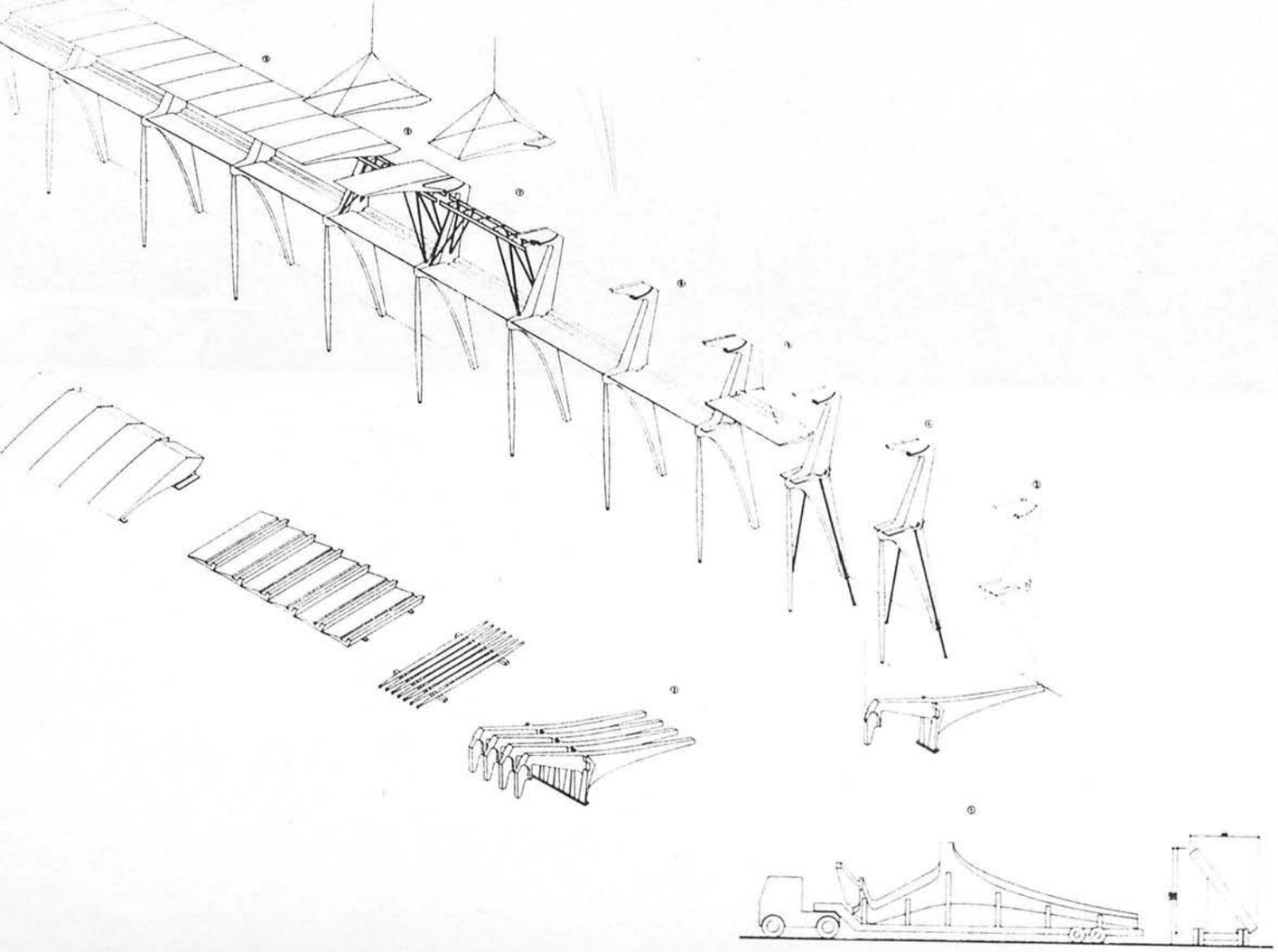




Running against the pre-existing building is a glazed, transparent zone that provides illumination to the work area and a richly varied view from the spaces situated along the exterior facada, and vice versa, revealing the geometry of its own static system, and preserving the continuity of the earlier facade. The cantilevered box beam portion of the canopy, with a wing-shaped section and covered with aluminium, marks a clear delineation between the stationary and moving zones. This enigmatic, effortlessly resolved composition, which places the opaque, visually heavy, portions of the canopy away from its anchor, and the visually light at the anchor, tends to accentuate the sense of flight of the cantilever. The resulting form strengthens the representative character of the PTT building.

The effectiveness of this structural system, formed by the addition of three different elements, is made possible only through their cooperative action. The joints and the support and bearing members work together with such continuity that they follow the ideal line of support.

Fotografías:
Paolo Rosselli



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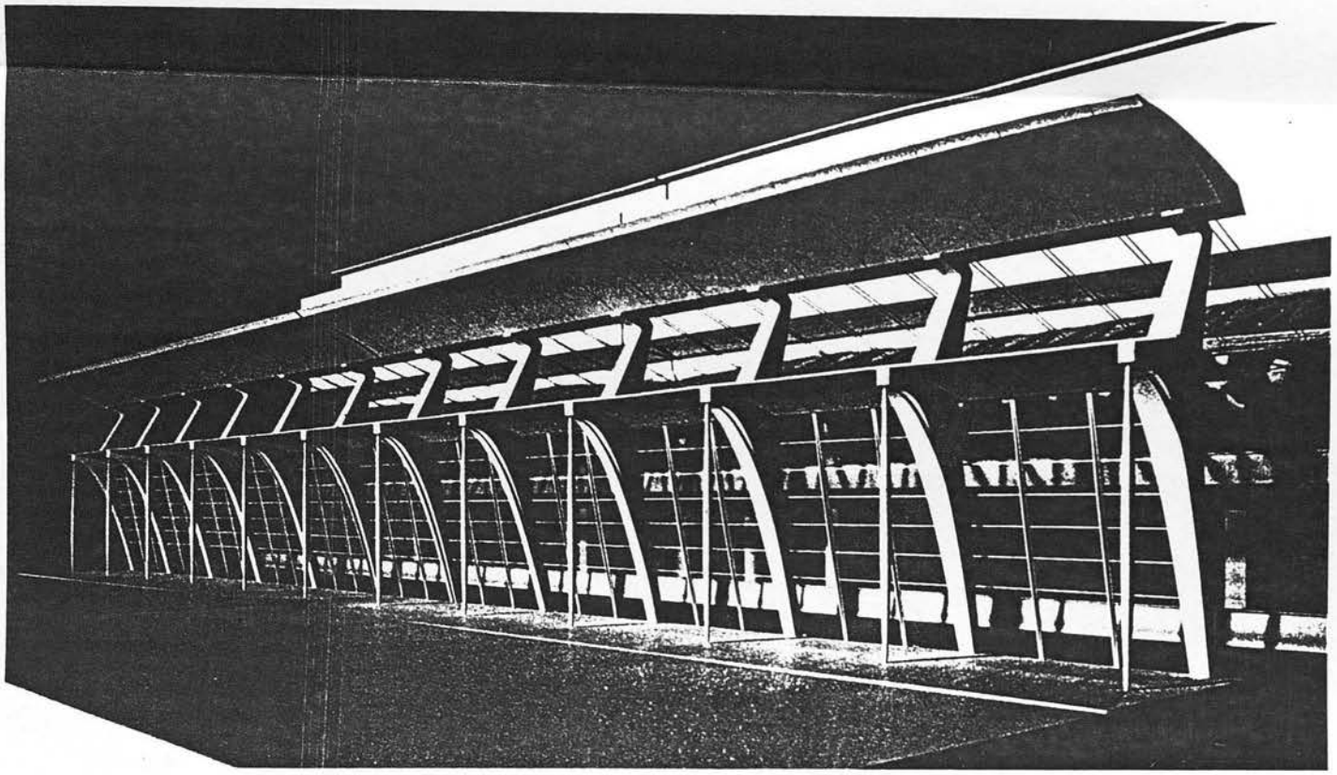
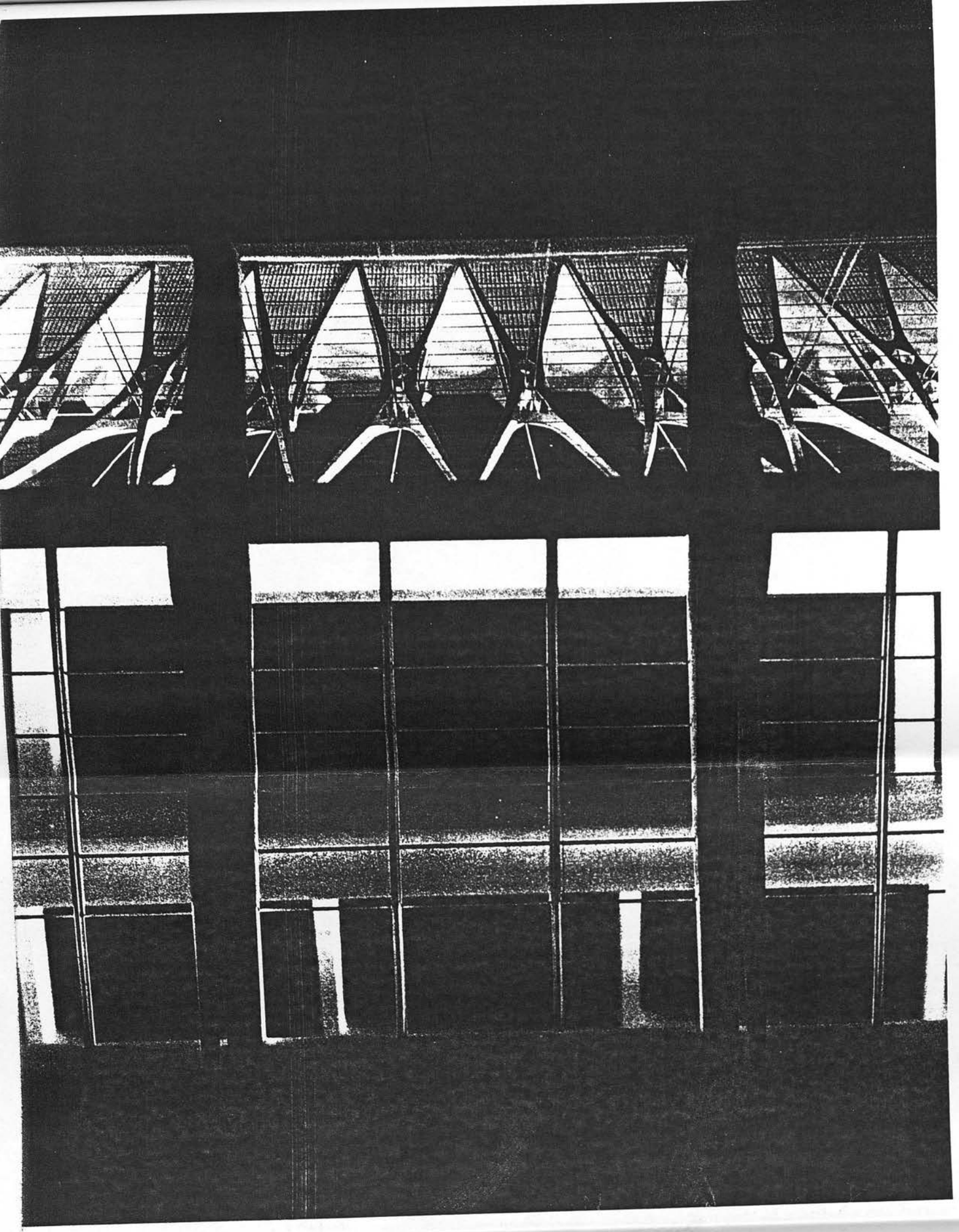
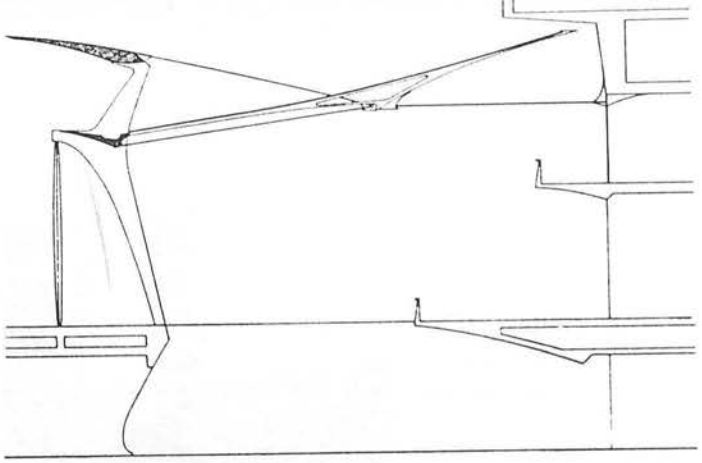
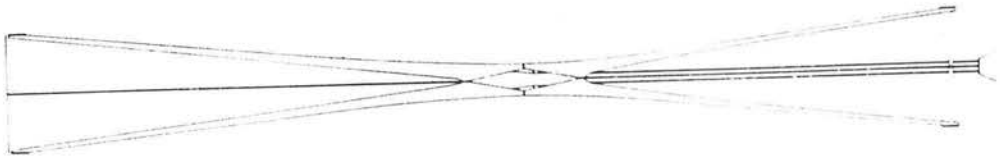




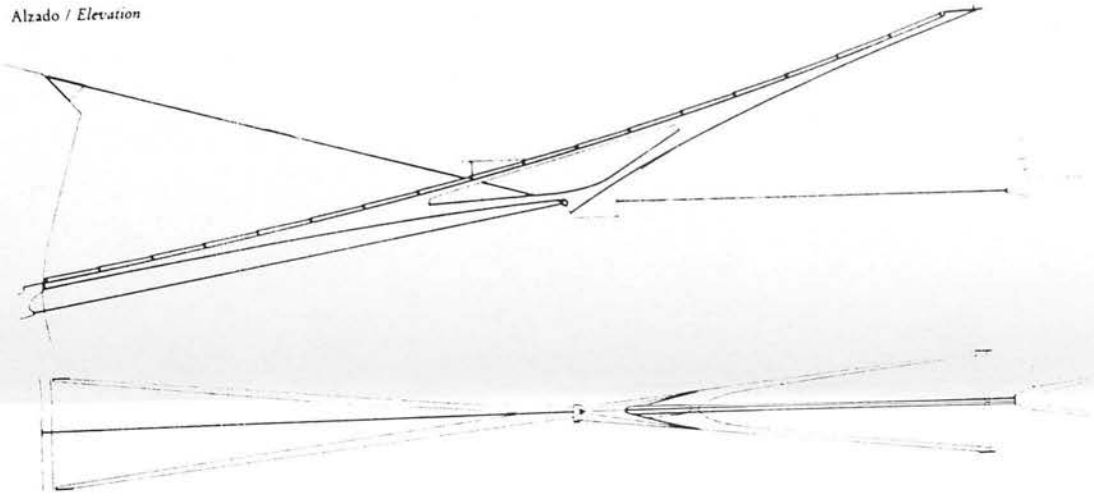


Sección transversal / *Cross section*

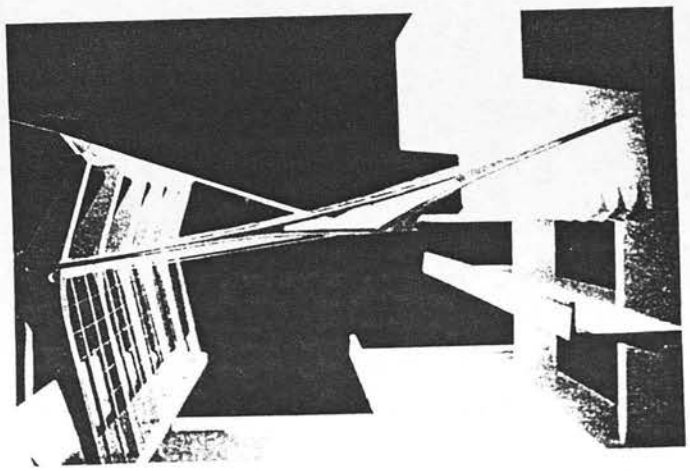


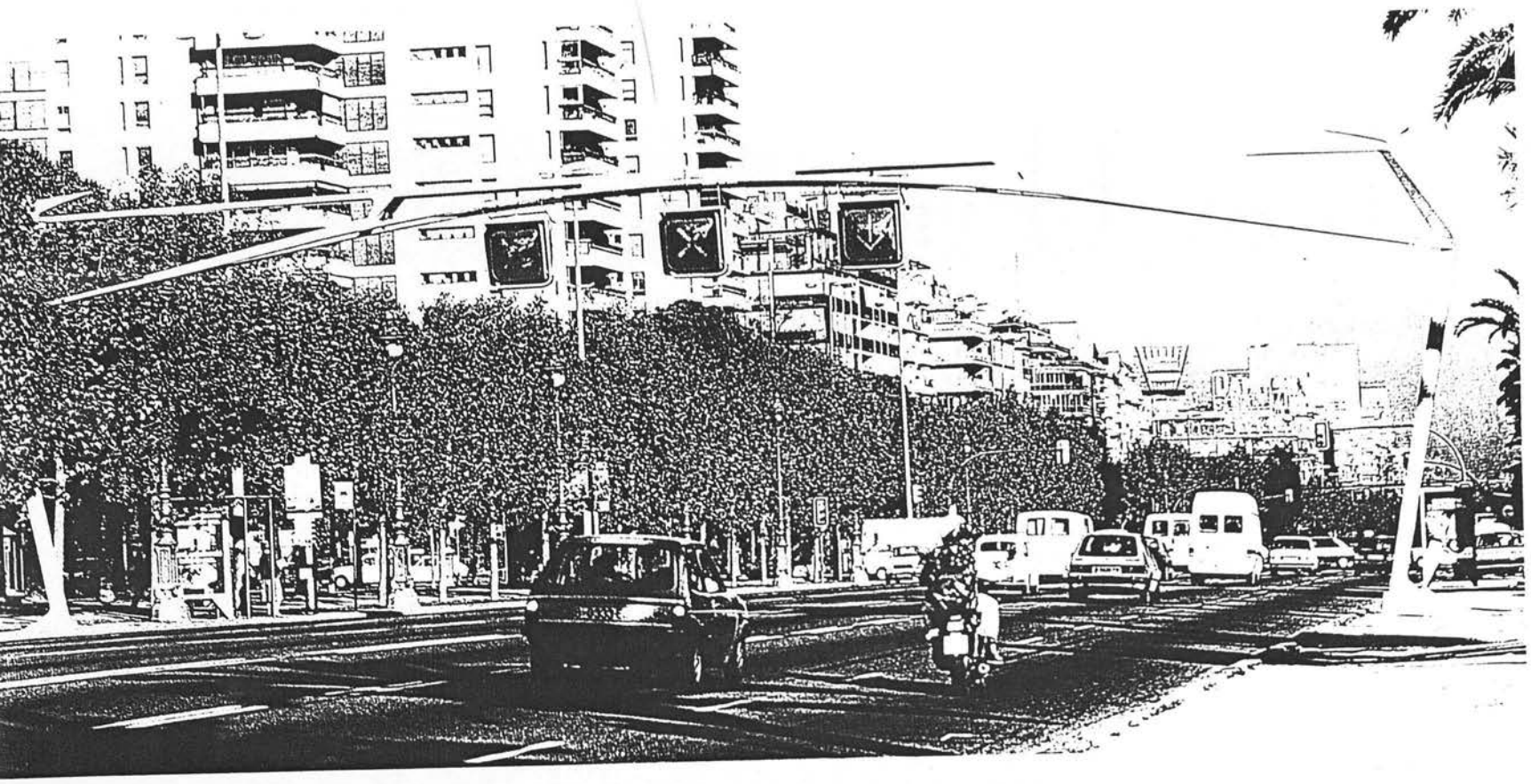
Vista superior / *View from above*

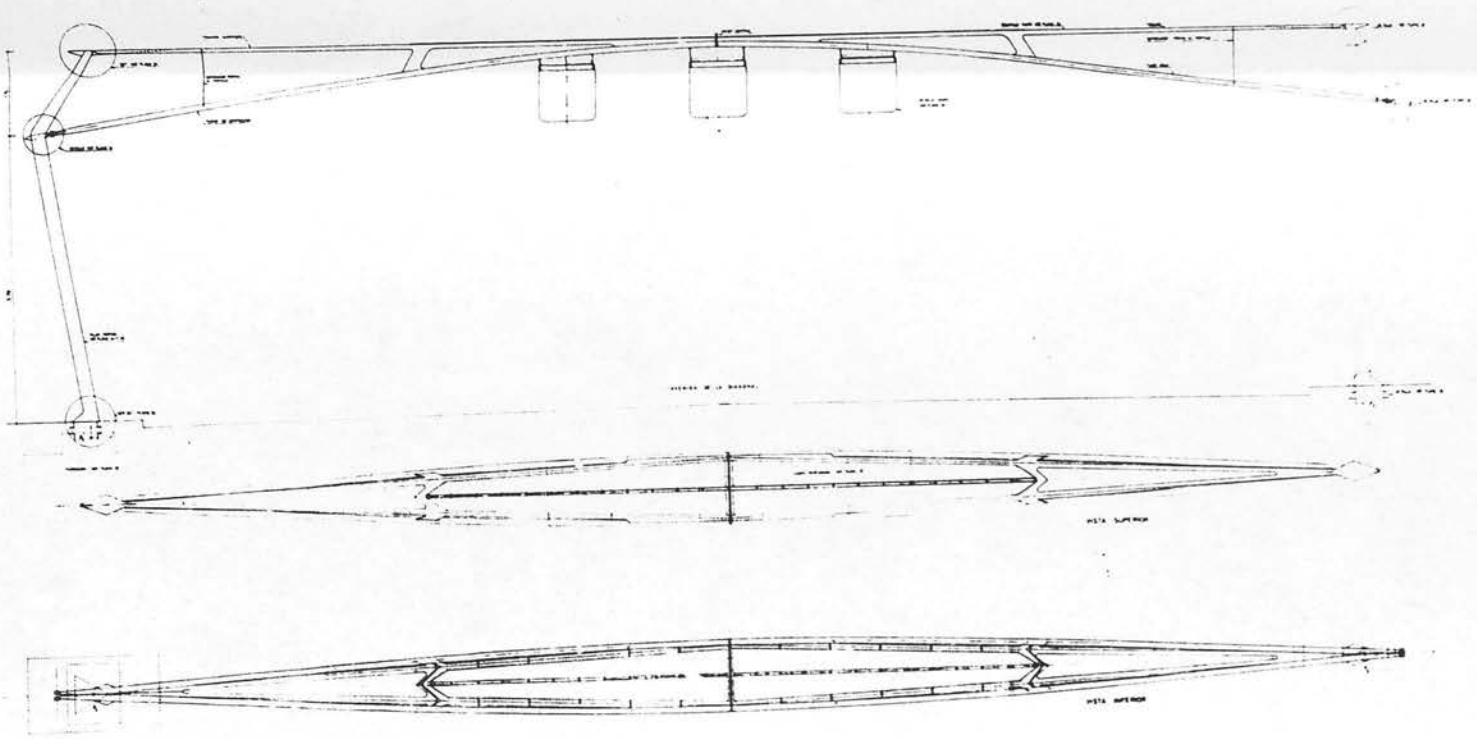
Alzado / *Elevation*



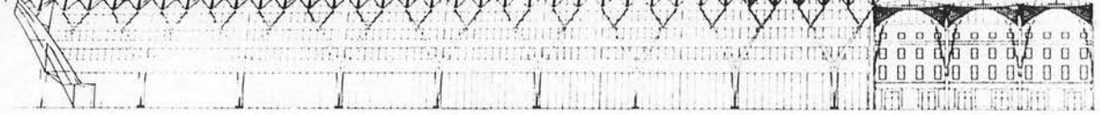
Vista inferior / *View from below*



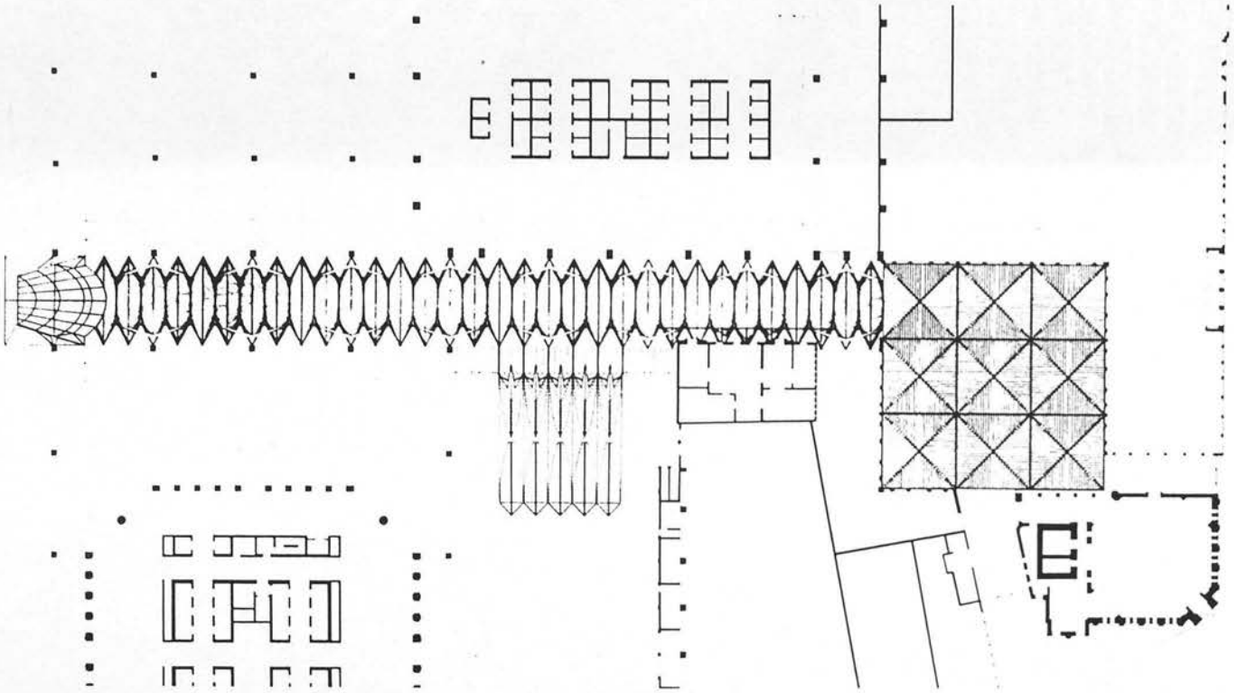




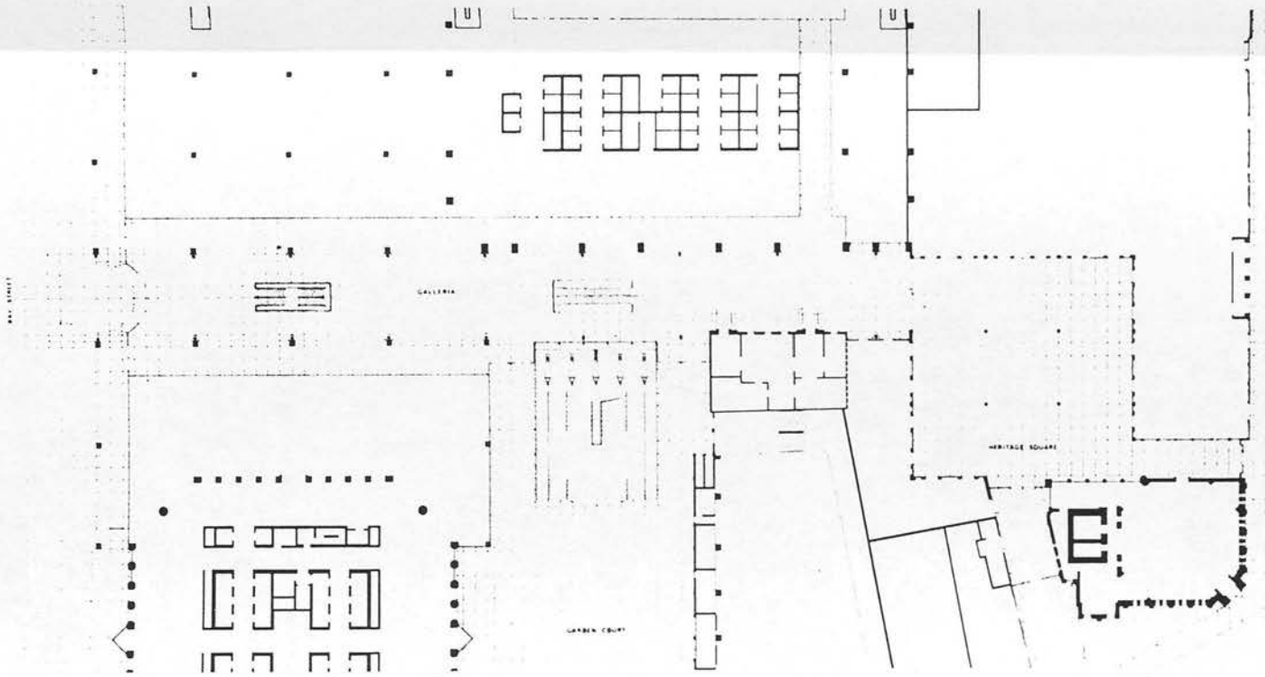
TRAFFIC LIGHT Alzado y vistas superior e inferior / *Elevation and views from above and below*



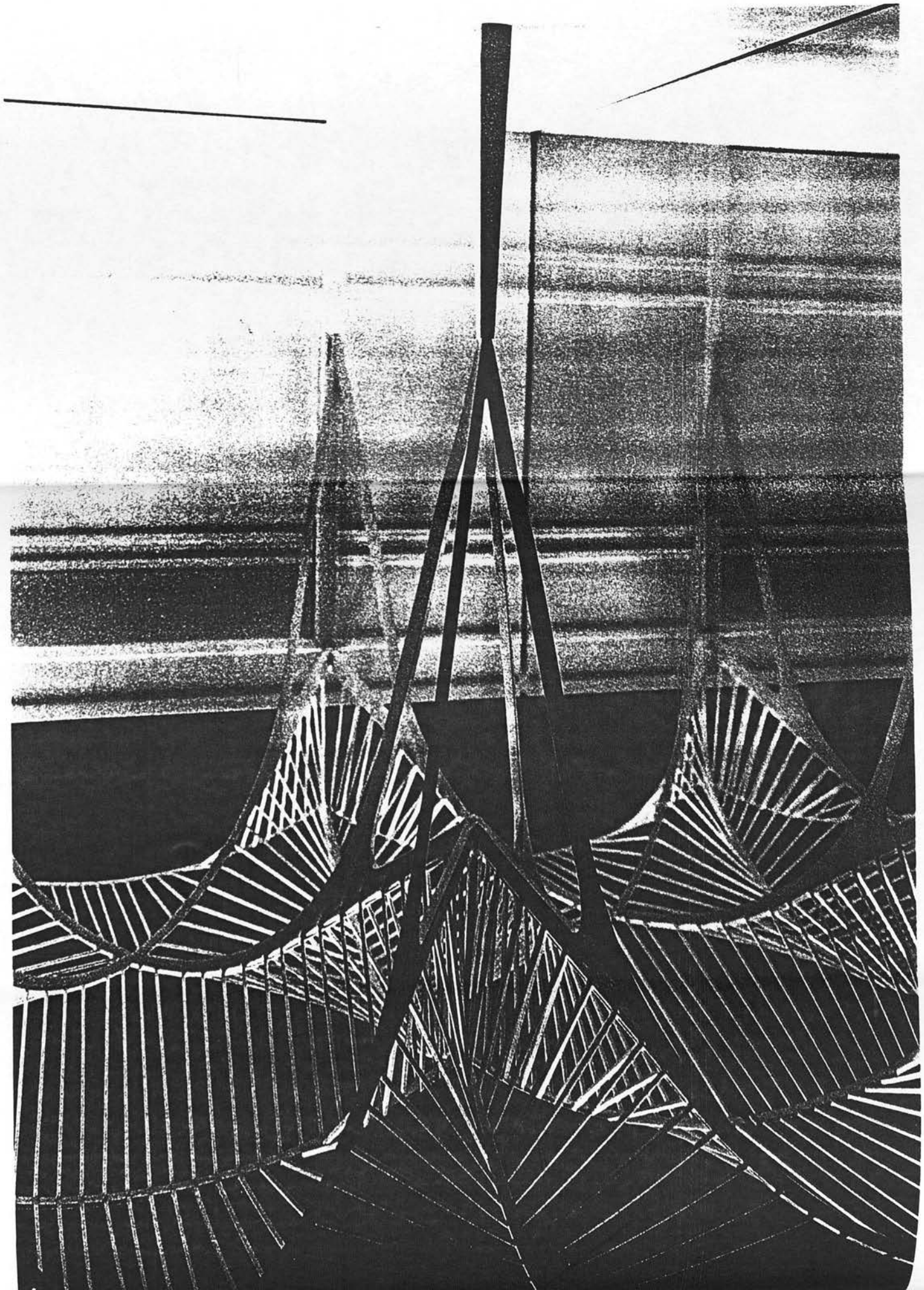
Sección longitudinal / *Longitudinal section*

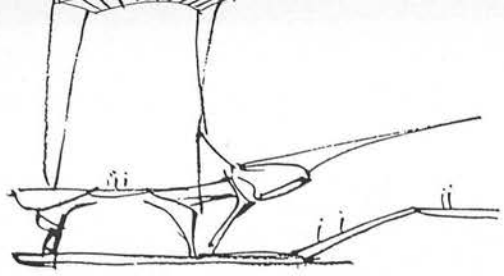


Planta de cubiertas / *Roof plan*

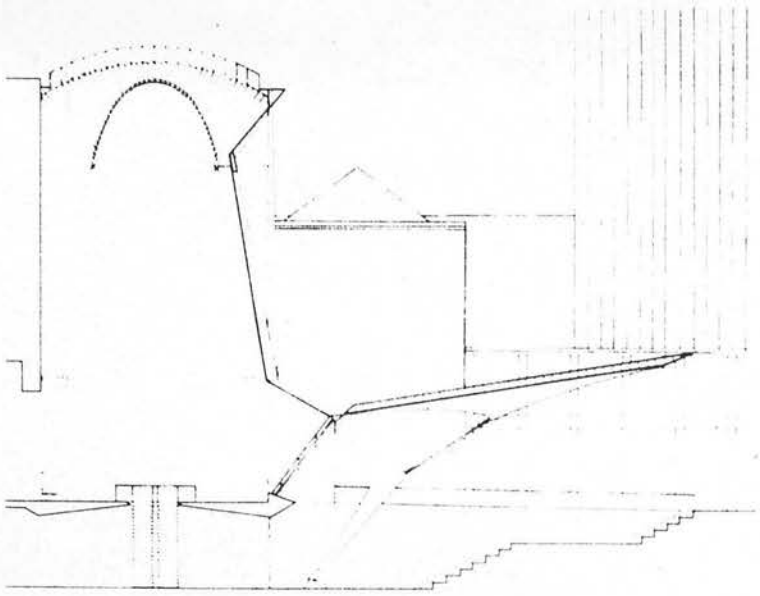


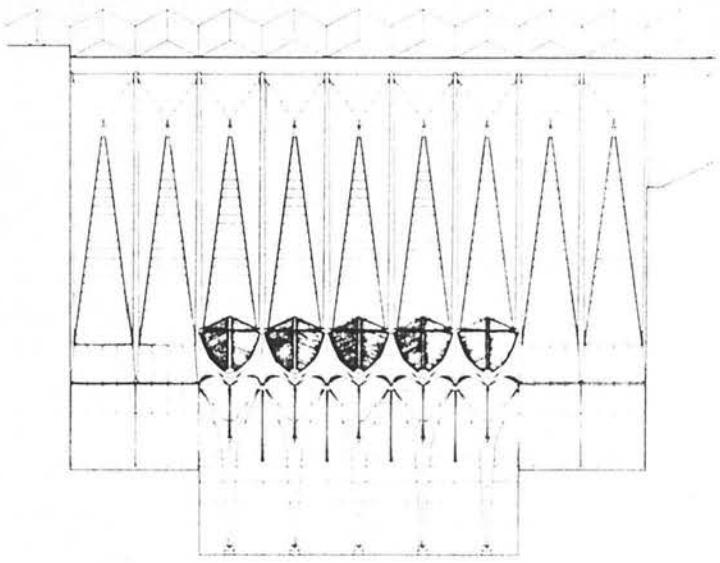
Planta de situación / *Site plan* **STREET GALLERY**





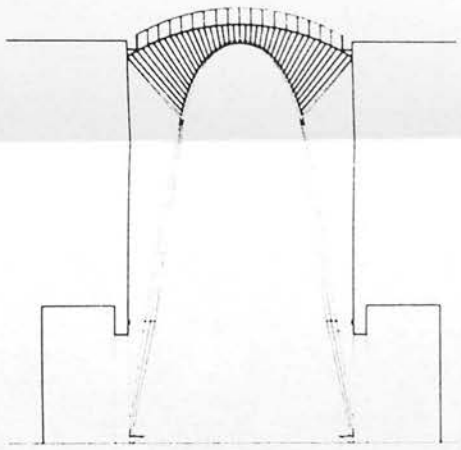
tuados a nivel peatonal—, que sirven como apoyo a las vigas curvadas de la cubierta. La línea de las puertas de entrada se encuentra en recesión con respecto al plano exterior de la fachada, originando así una marquesina cónica, y una especie de pequeña plaza de acceso.

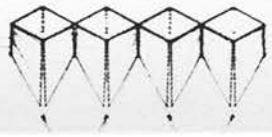


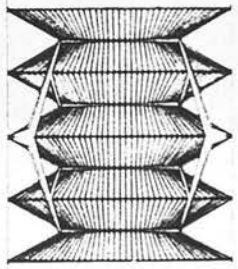


Arriba:
Acceso al patio-jardín
Croquis, sección transversal y alzado
Above:
Access to garden court
Rough sketch, cross section an elevation

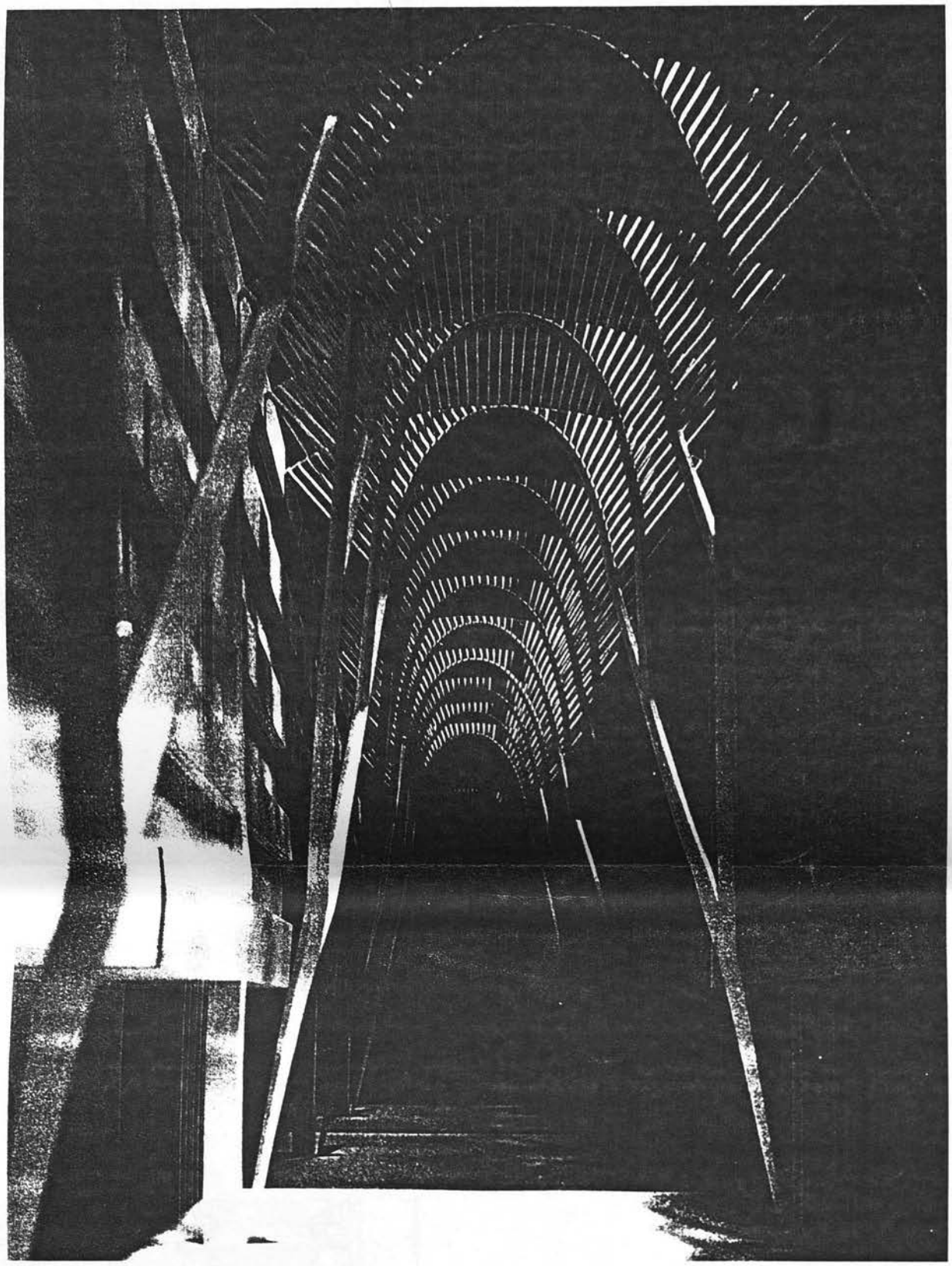
Abajo:
Galería. Croquis, planta de cubiertas, sección transversal y alzado estructura
Below:
Gallery. Rough sketch, roof plan, cross section and elevation





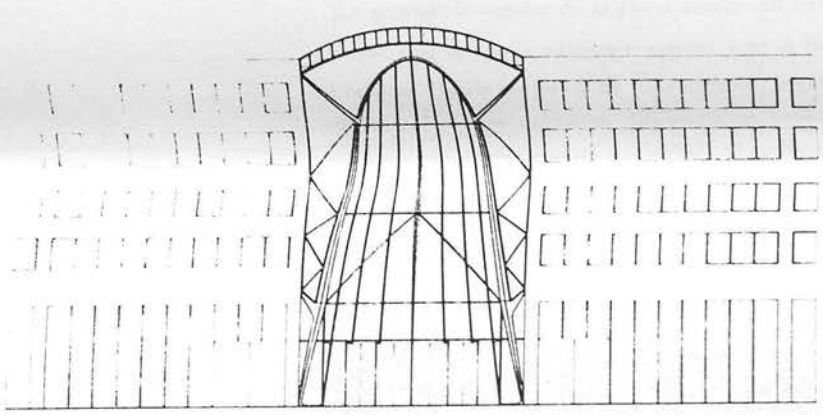


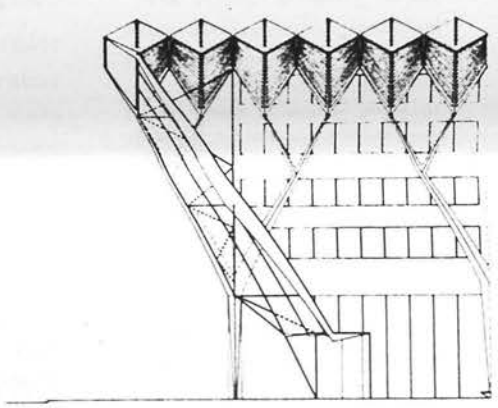
The gallery consists of inclined and ramified supports —placed on the pedestrian level—, which bear the curved roof supporting beams. The line of entrance doors is drawn in from the outward plane of the façade, thus creating a conical canopy and a kind o little entrance square.

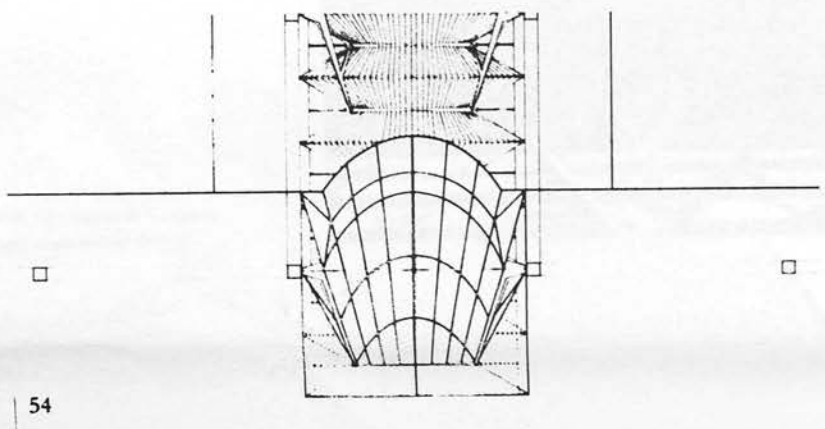


Vista transversal de la Galeria / *View across the gallery*

Vista de la estructura de la Plaza Heritage / *View of the structure of Heritage Square*





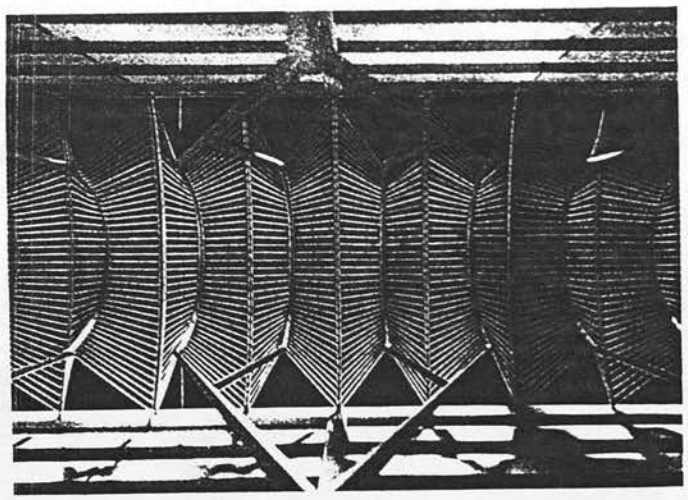


Marquesina de acceso desde Bay Street
Secciones horizontal, transversal y longitudinal
Access from Bay Street. Canopy
Horizontal, cross and longitudinal sections

Plaza Heritage, vista superior / *Heritage Square, view from above*

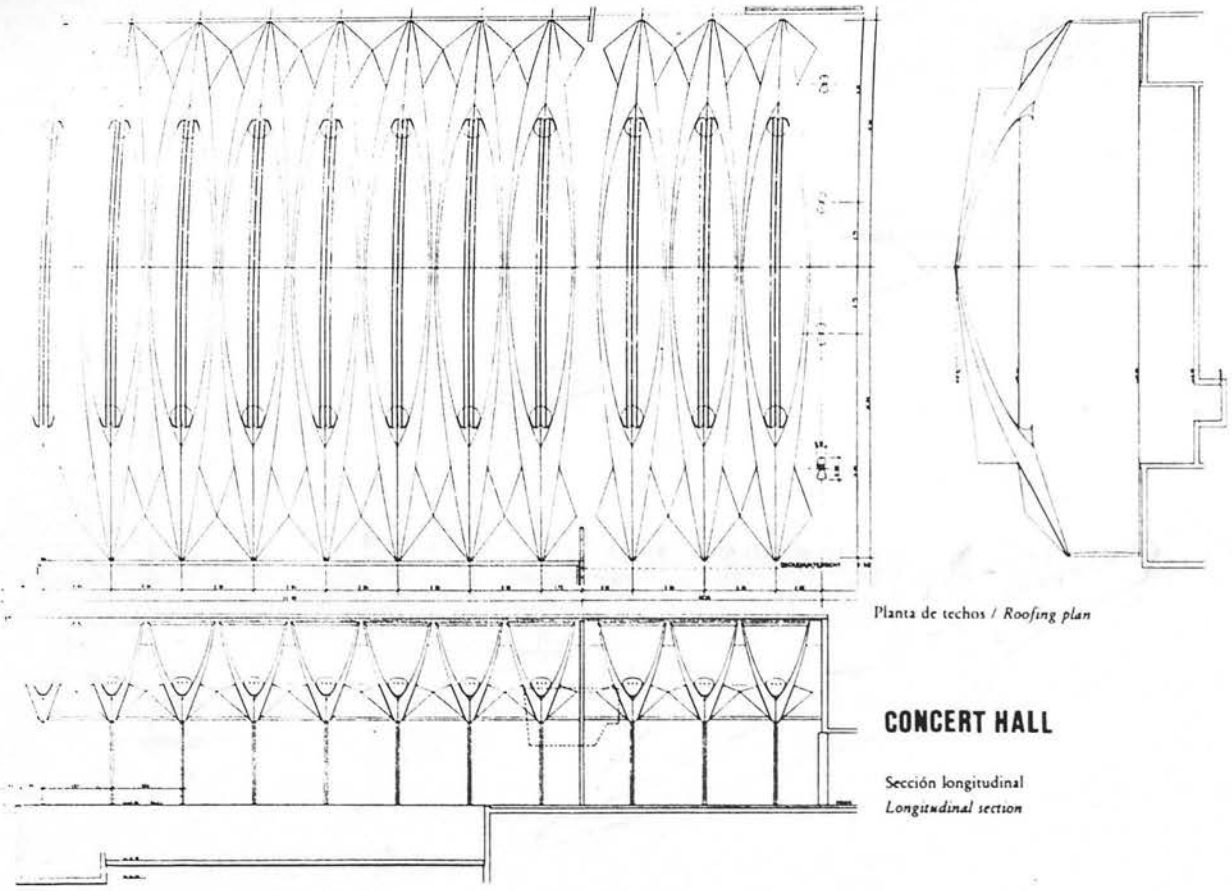
La geometría regular de la *plaza* admite un sistema geométrico central, que se apoya en cuatro puntos y en la fachada que rodea el recinto. Desde un punto de vista estático, la cubierta de la plaza constará de nueve unidades, dispuestas en zonas de 3 × 3. Cada zona comprende dos arcos, que se cruzan en diagonal, con soporte en las cuatro esquinas. A media altura, los soportes se dividen en cuatro vigas transversales, de forma arbolada, que se corresponden así con la estructura de la galería.

The regular geometry of the square admits a central geometrical system, which rests on four supports as well as on the enclosing façade. From a statical point of view, the roofing of the square will consist of four units. These will be arranged in 3 × fields. Each field consists of cross-linking bows, which are supported on its four corners. At the middle height, the supports divide in four crossbeams, having a tree-like shape, corresponding to the structures of the gallery.



Galería, vista inferior de la cubierta
Gallery, inside view of the roof

Fotografías:
Heinrich Helfenstein

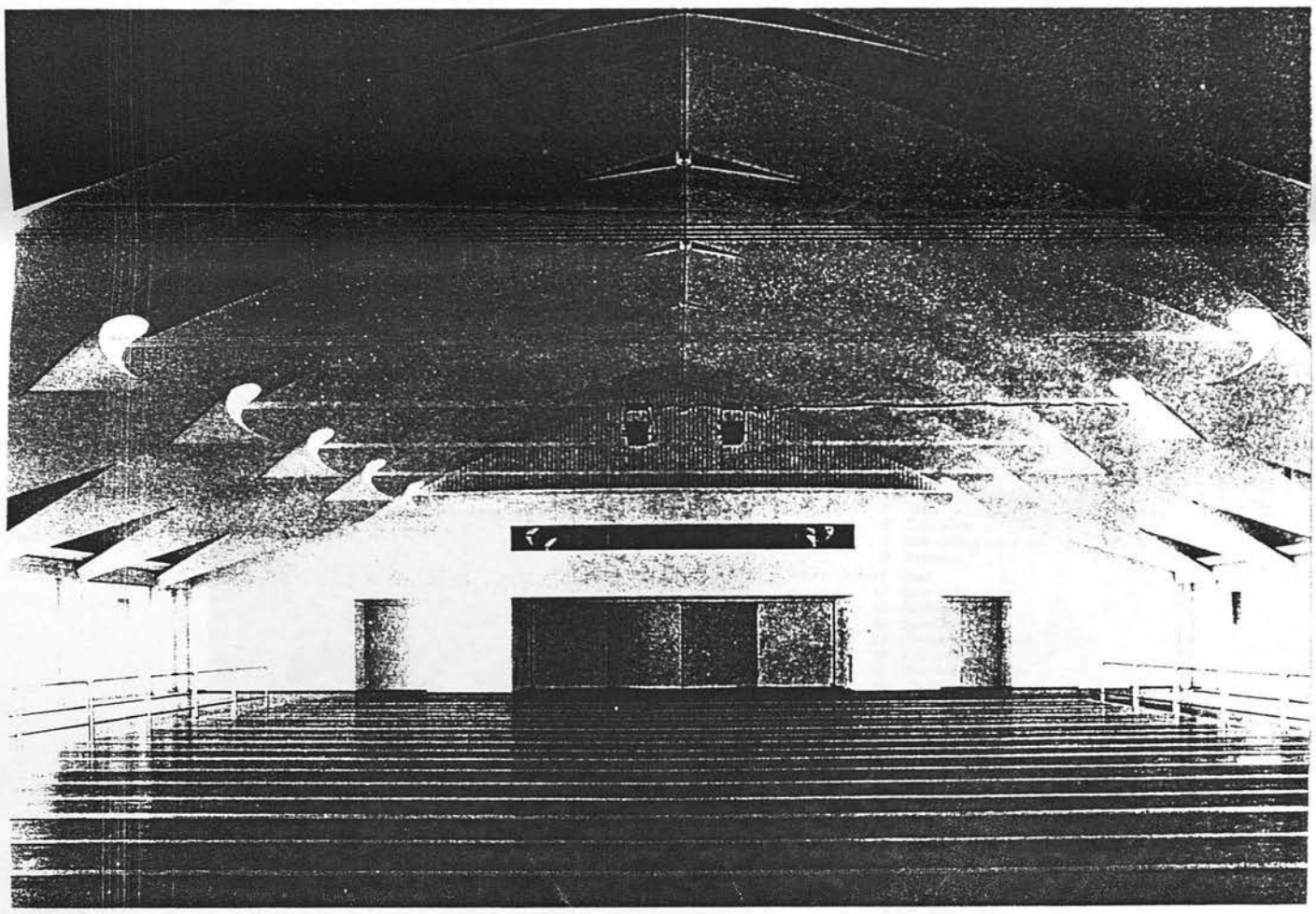


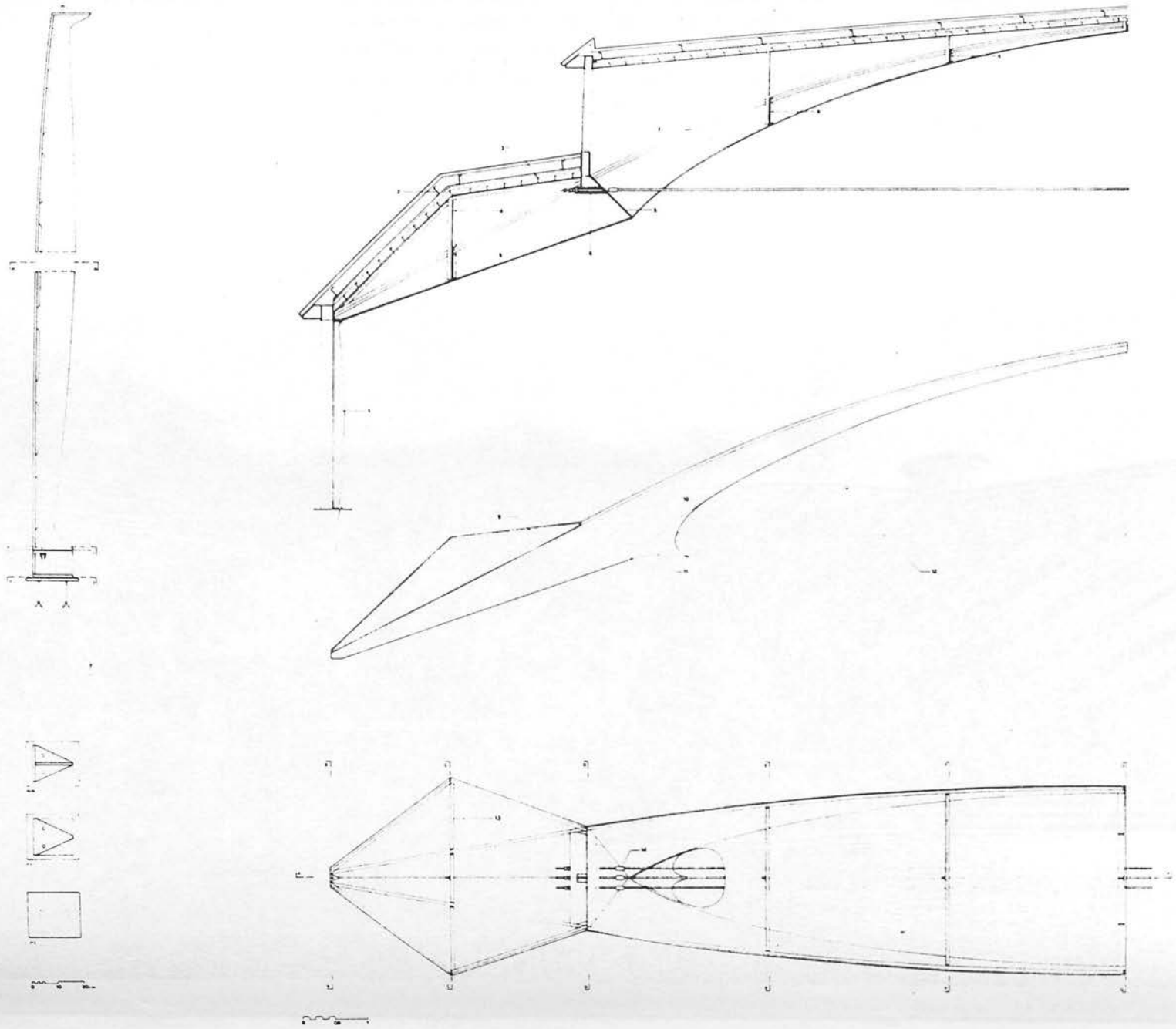
Sección transversal
Cross section

Planta de techos / *Roofing plan*

CONCERT HALL

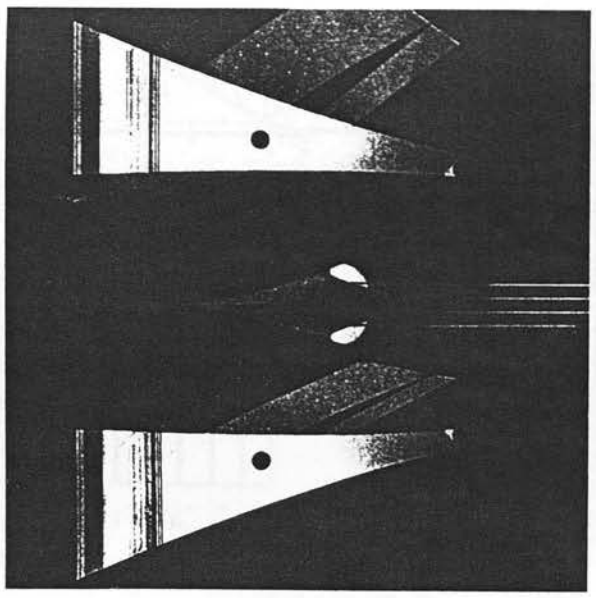
Sección longitudinal
Longitudinal section





Cubrición y jácena. Detalles

1 Soporte.
2 Perfil en Z.
3 Casetones tipo «Montaña» o similar.
4 IPE 120.
5 e = 5 mm.
6 Rigidizador transversal (e = 20, e = 12).
7 Tirante.
8 Chapa de borde.
9 Revestimiento lateral e = 2 mm.
10 Estructura.
11 Pico.
12 Tensor 0 30 mm.
13 Rigidizador.
14 Husos.
15 Angular 80 × 10 mm.
16 Revestimiento de chapa.
17 Rigidización transversal.
18 Presilla.
19 Angular 50 × 5 mm.
20 Angular de borde 50 × 5 mm.



Roofing and girer. Details

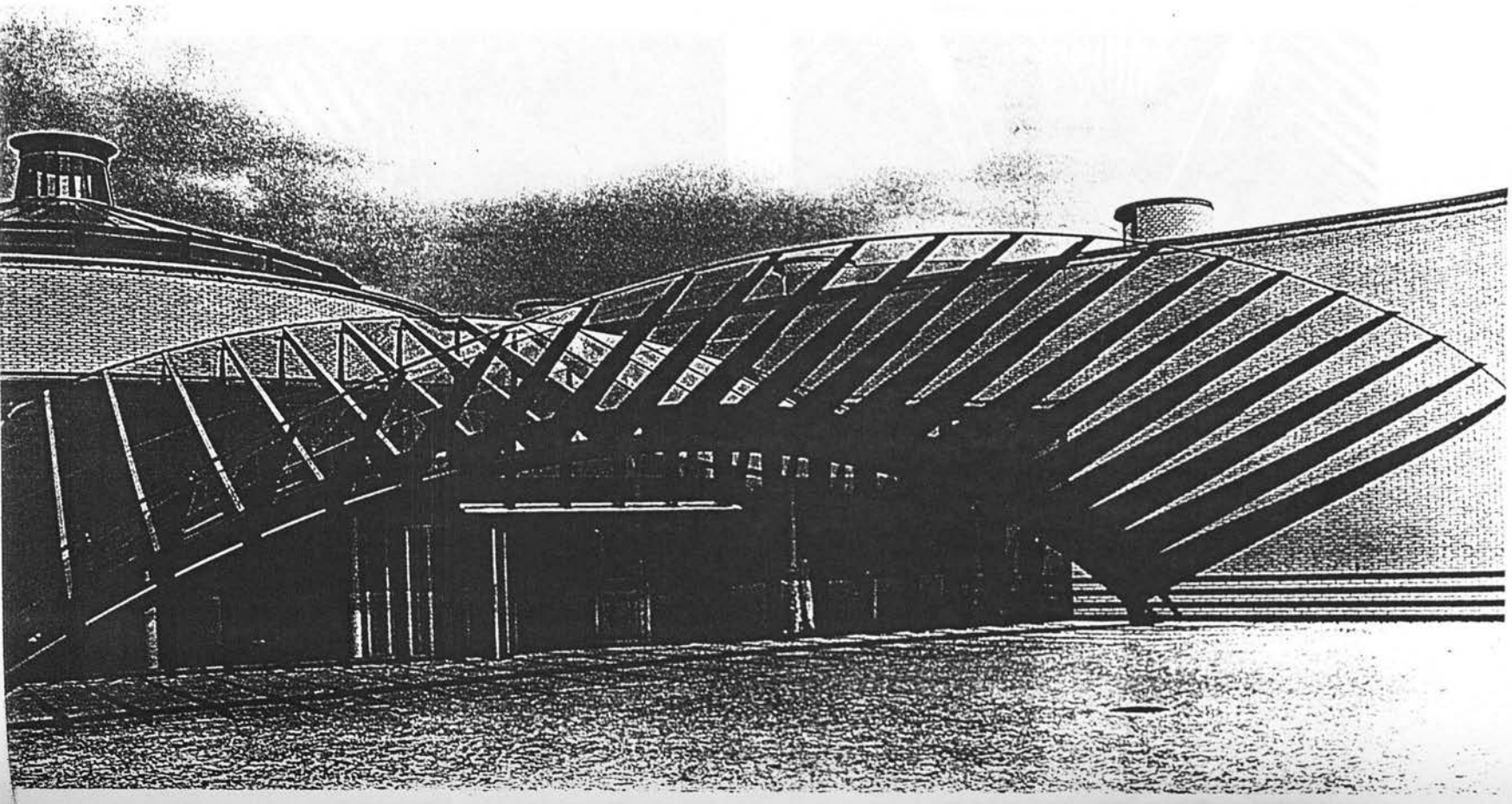
1 Support.
2 Z-profile.
3 «Montana» coffering or similar.
4 IPE 120.
5 t = 5 mm.
6 Transverse stiffener (t = 20, t = 12).
7 Brace.
8 Edge plate.
9 Side coating t = 2 mm.
10 Structure.
11 Peak.
12 Strut 0 30 mm.
13 Stiffener.
14 Spindles.
15 Corner 80 × 10 mm.
16 Foil coating.
17 Transverse stiffener.
18 Cramp.
19 Corner 50 × 5 mm.
20 Edge profile 50 × 5 mm.

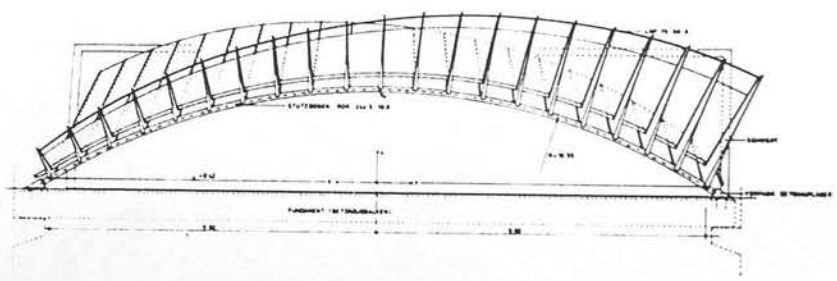
1 *Marquesina*

La forma de la *marquesina de acceso* se genera a partir de la intersección de dos superficies cónicas que forman un arco. La estructura está compuesta por un arco de acero de sección triangular, al cual se fijan las vigas en voladizo que sostienen los paneles de cristal.

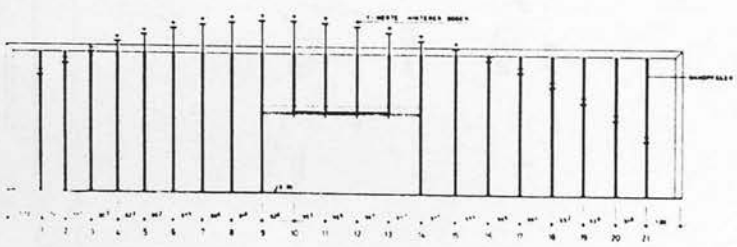
WOHLEN SCHOOL

The form of the *entrance canopy* was generated by two conical surfaces, whose intersection forms an arch. The arched steel spine is a triangular girder to which the cantilevered ribs supporting the glass panels are affixed.

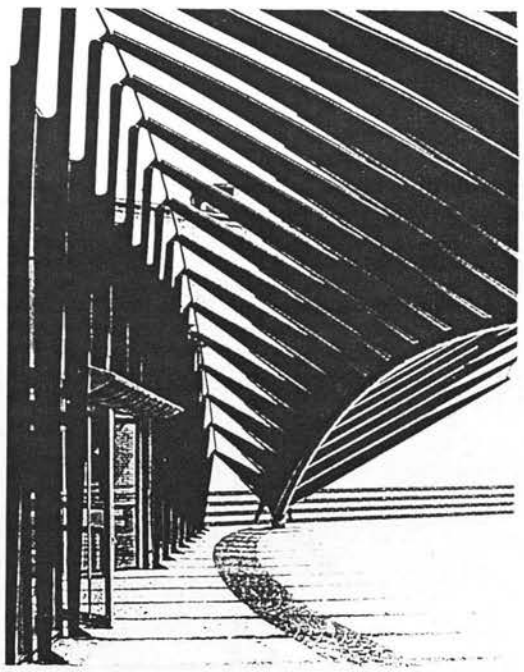




Alzado frontal / *Front elevation*



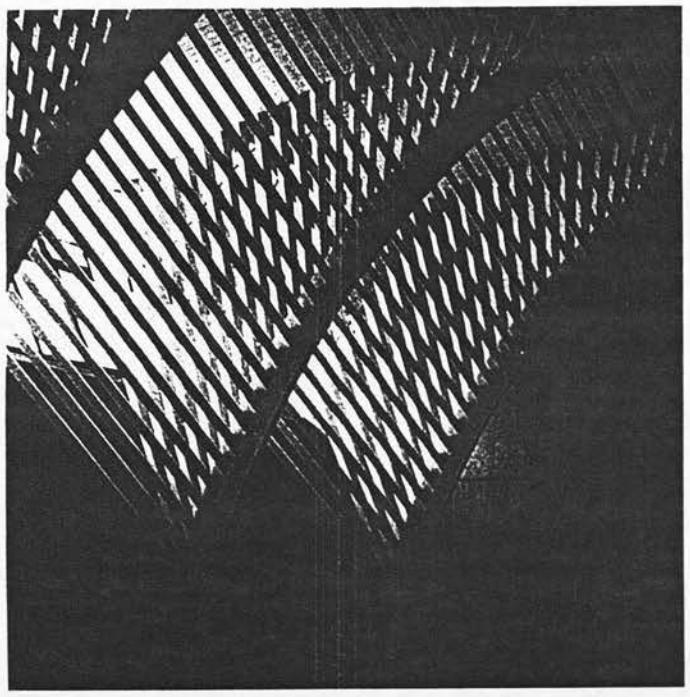
Alzado de la estructura de fachada / *Structure elevation of the facade*

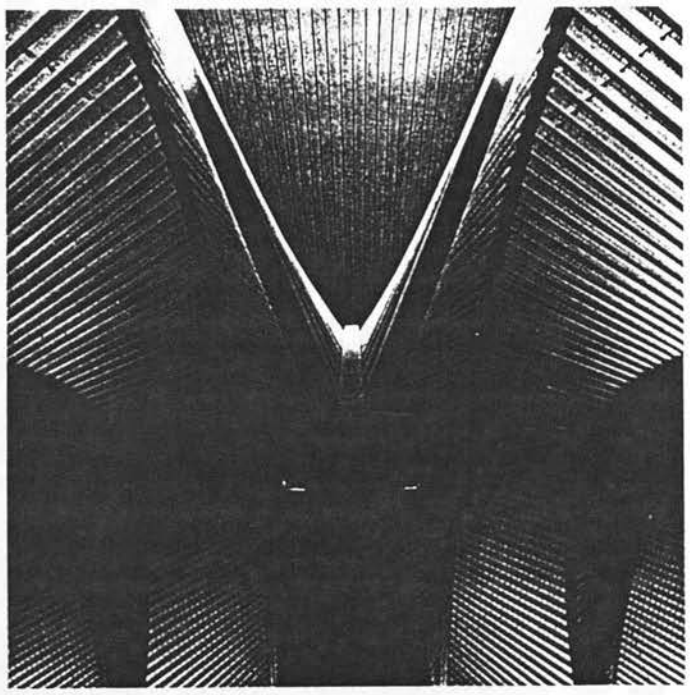


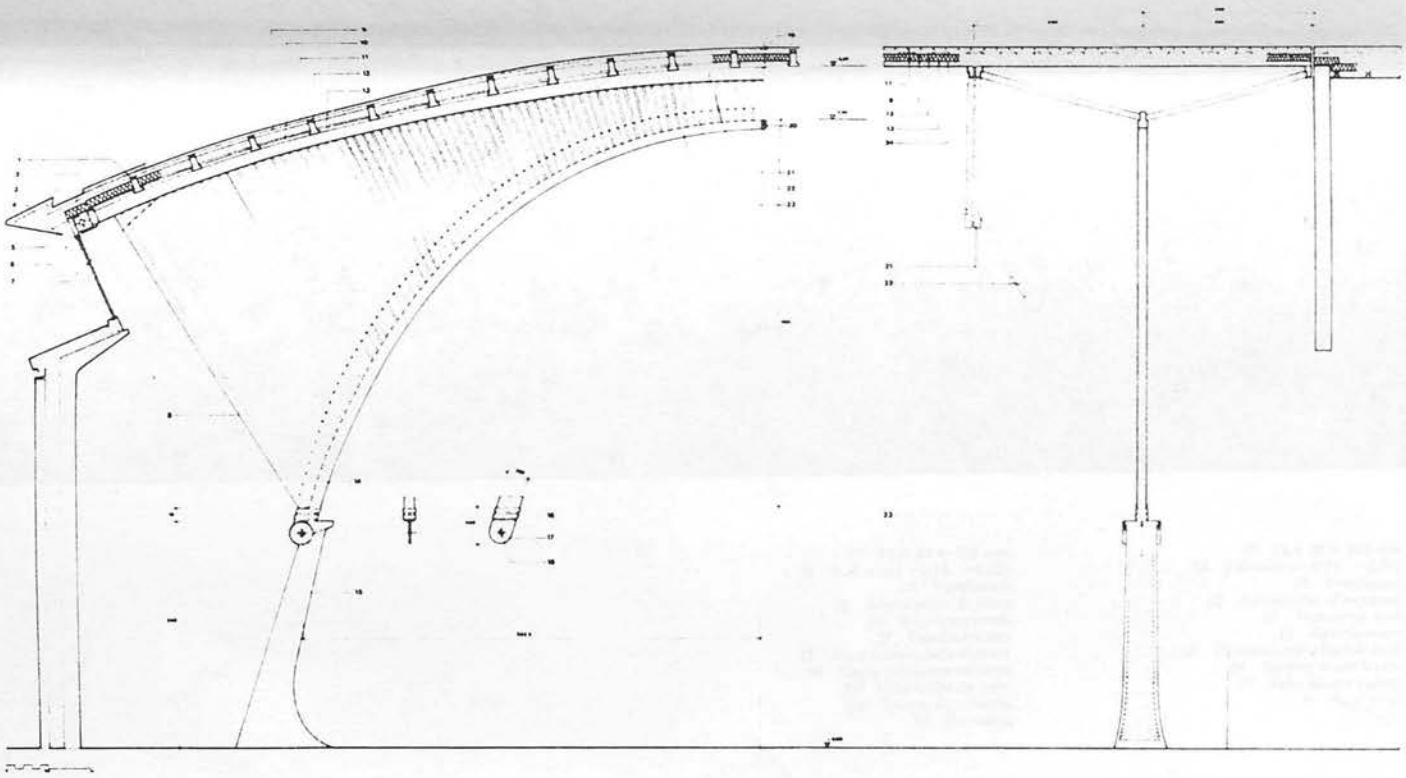
4 Atrio

El *atrio* presenta la imagen de una gran carpa, un punto de encuentro para estudiantes. Está construido con vigas de madera laminada, cables y elementos de acero, y recubierto con cristal translúcido. Este elemento fue específicamente diseñado para que pudiera verse, y sentirse, el flujo de las fuerzas que atraviesan la estructura.

The *entrance hall* presents the image of a great tent, a meeting point for the students. It is contructed from laminated-wood girders, cables and steel spindles, and sheathed with opaque glass. This element was specifically designed so that one could see, and feel, the flow of the forces through it.





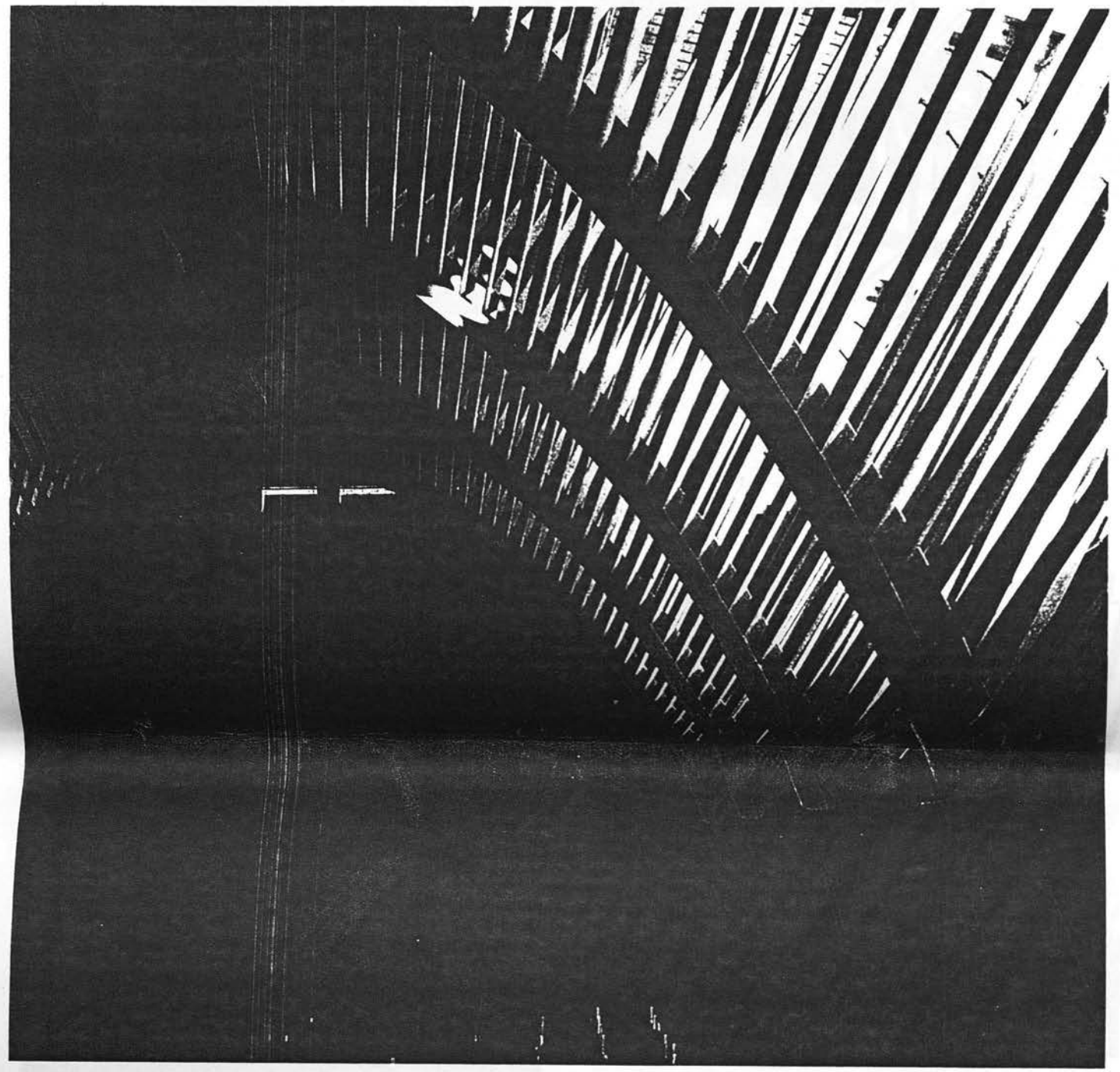


En la página izquierda:
Secciones longitudinal y transversal

1 Correa.
2 Angular metálico.
3 Ranura.
4 Cabrio en punta.
5 Alero de chapa perforada.
6 Correa de remate.
7 Radio r = 1747 cm.
8 Madera de borde.
9 Ranura de ventilación.
10 Capa inferior de cubierta.
11 Capa de cubierta.
12 Armadura (correa) longitudinal encolada.
13 Aislamiento.
14 Armadura principal de madera laminada encolada.
15 Soporte de hormigón.
16 FLA 10 × 40 mm.

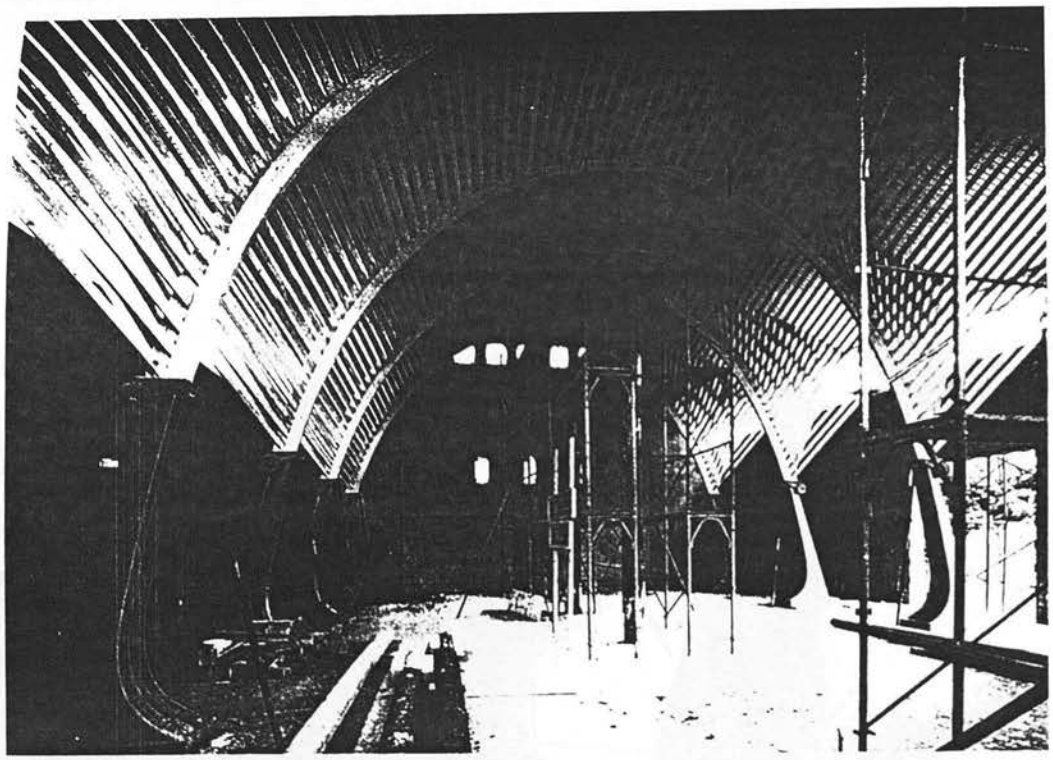
On the left page:
Longitudinal and cross sections

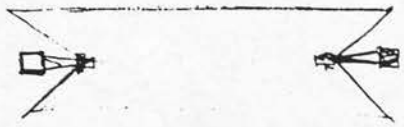
1 Strap.
2 Metal profile.
3 Slot.
4 Pointed post.
5 Perforated plate gable-end.
6 Topping strap.
7 Radius r = 1747 cm.
8 Edge wood.
9 Ventilation slot.
10 Lower layer of roof.
11 Layer of roof.
12 Glued longitudinal frame.
13 Isolation.
14 Main frame of glued sheet wood.
15 Concrete support.
16 FLA 10 × 40 mm.

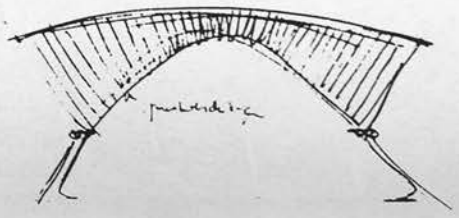


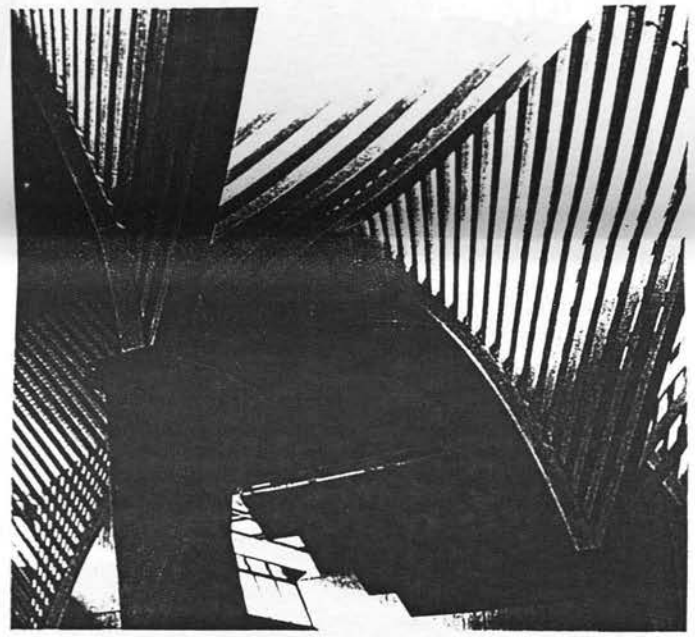
17 FLA 20 × 280 mm.
18 0 30 mm (−0,15, −0,25).
19 Ventilación.
20 Articulación de clave.
21 Arco transversal.
22 Ramificaciones.
23 Arco triarticulado elíptico.
24 Capas inferiores cubierta.
25 Falso techo de yeso.
26 Correa de cubierta.
27 Escenario.

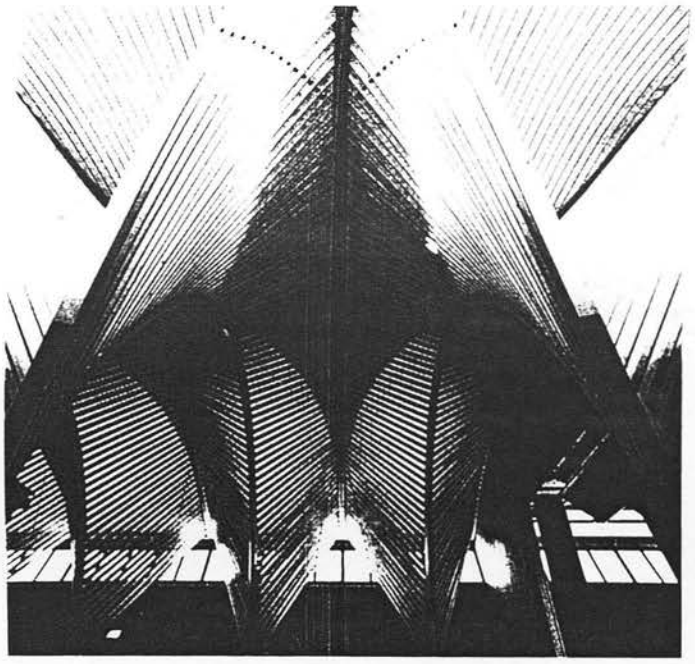
17 FLA 20 × 280 mm.
18 0 30 mm (−0,15, −0,25).
19 Ventilation.
20 Articulation of keystone.
21 Transverse arch.
22 Ramifications.
23 Triarticulated elliptical arch.
24 Roofing lower layers.
25 False plaster ceiling.
26 Roof strap.
27 Stage.



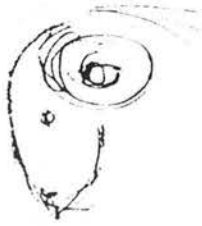


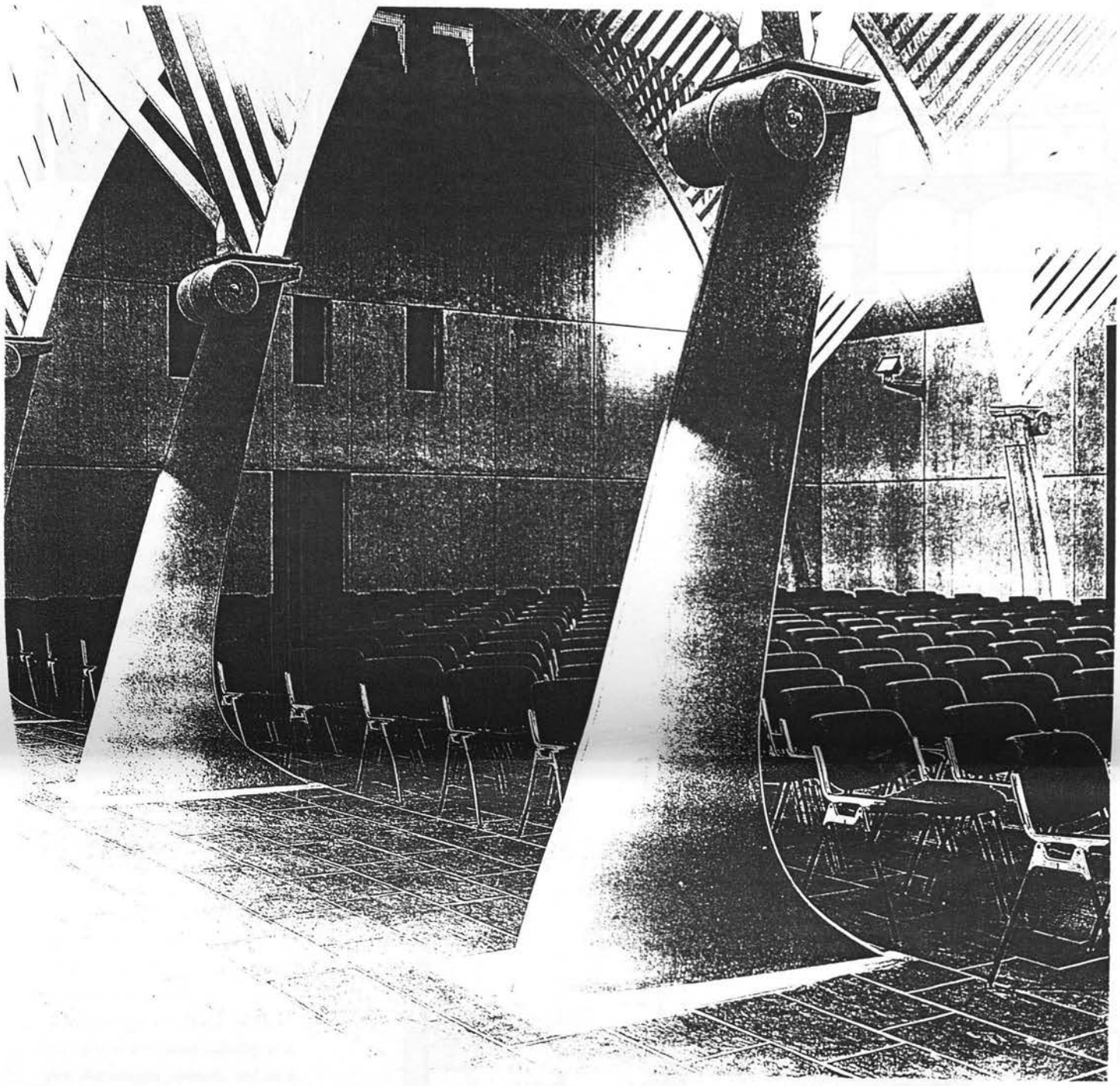


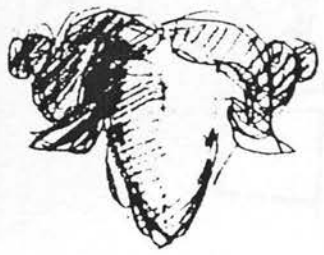


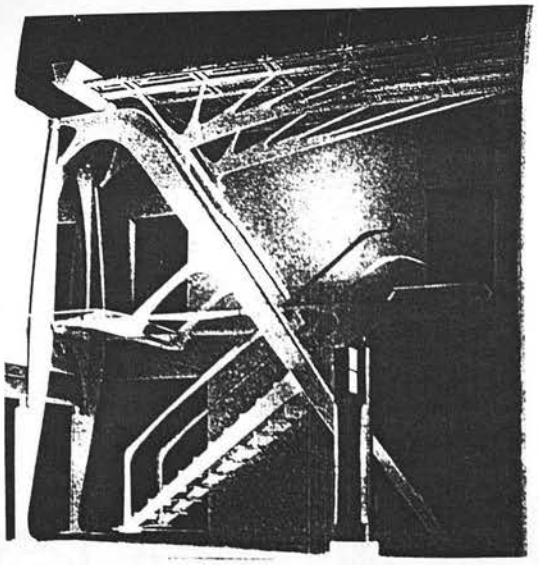


Fotografias:
Paolo Rosselli

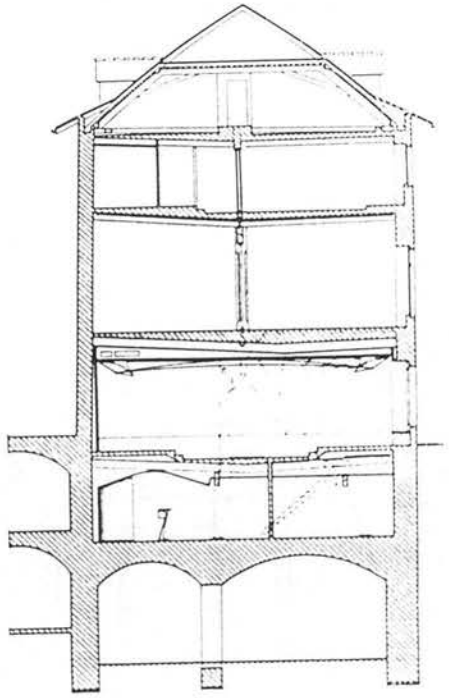








Maqueta
Model

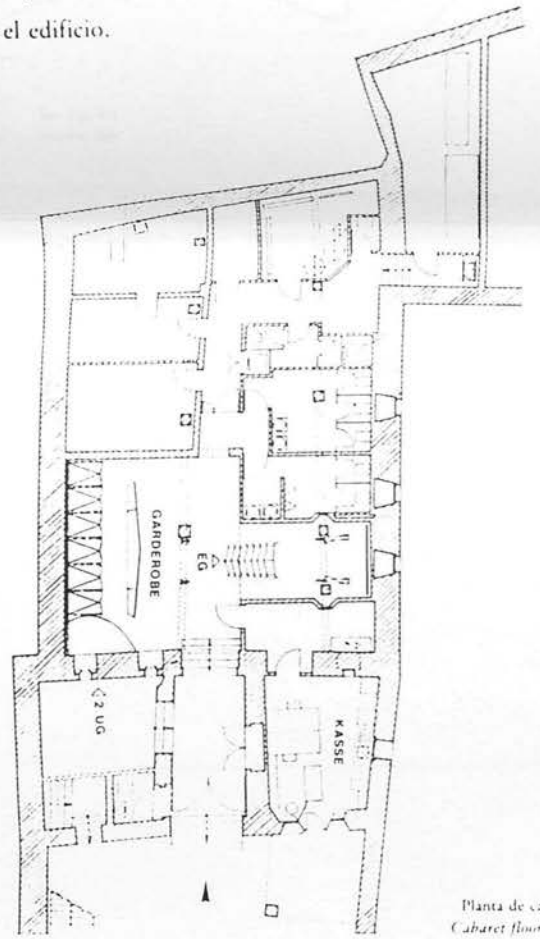


Sección transversal por escalera
Cross section through staircase

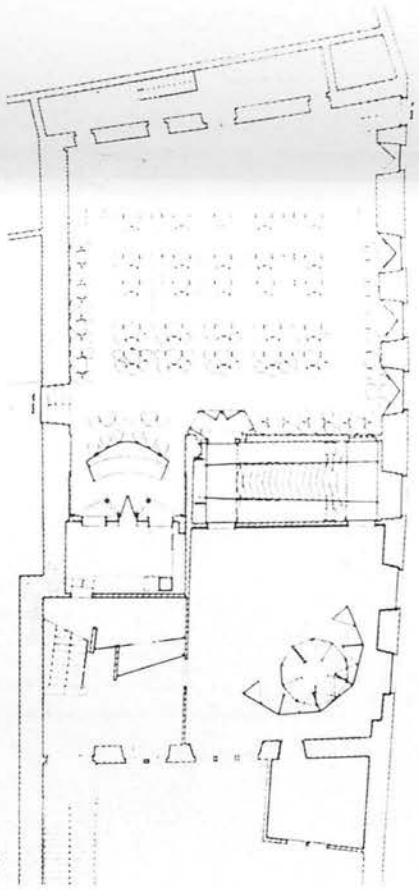
Como alternativa a las soluciones combinadas con las antiguas estructuras, que en cualquier caso hubieran supuesto la construcción de importantes estructuras nuevas, se propuso la idea de un *puente-escalera*, que así concentra las cargas en un único punto, de manera que puedan ser recogidas por un soporte sólido, esquivando las partes antiguas y evitando la necesidad de insertar elementos portantes pesados. De este modo, la escalera juega un doble papel: como nuevo elemento de distribución y como solución para el sostenimiento de la estructura de todo el edificio.

STAIRCASE STRUCTURE

As an alternative to solutions *mixed* with the old structures, which would in any case have entailed the building of major new structures, we came up with the idea of a *stair-bridge* which concentrates the loads into a single point so that they can then be taken up by a solid support, stepping over the ancient parts and avoiding any need to insert heavy bearing members. Thus, the stairway acts in a twofold capacity as a new distributive element, and as a bearing solution to the frame of the building as a whole.

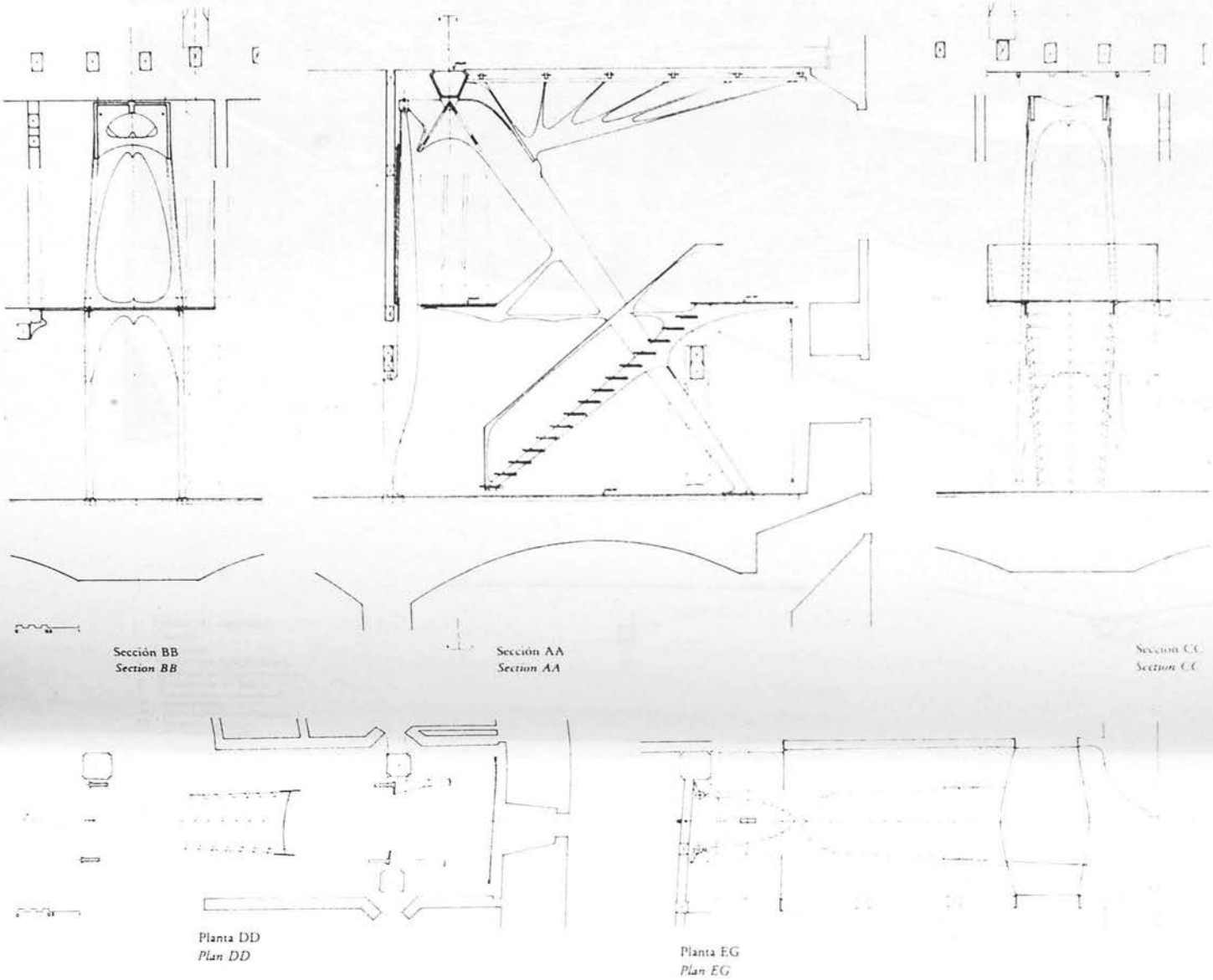


Planta de acceso
Entrance floor plan

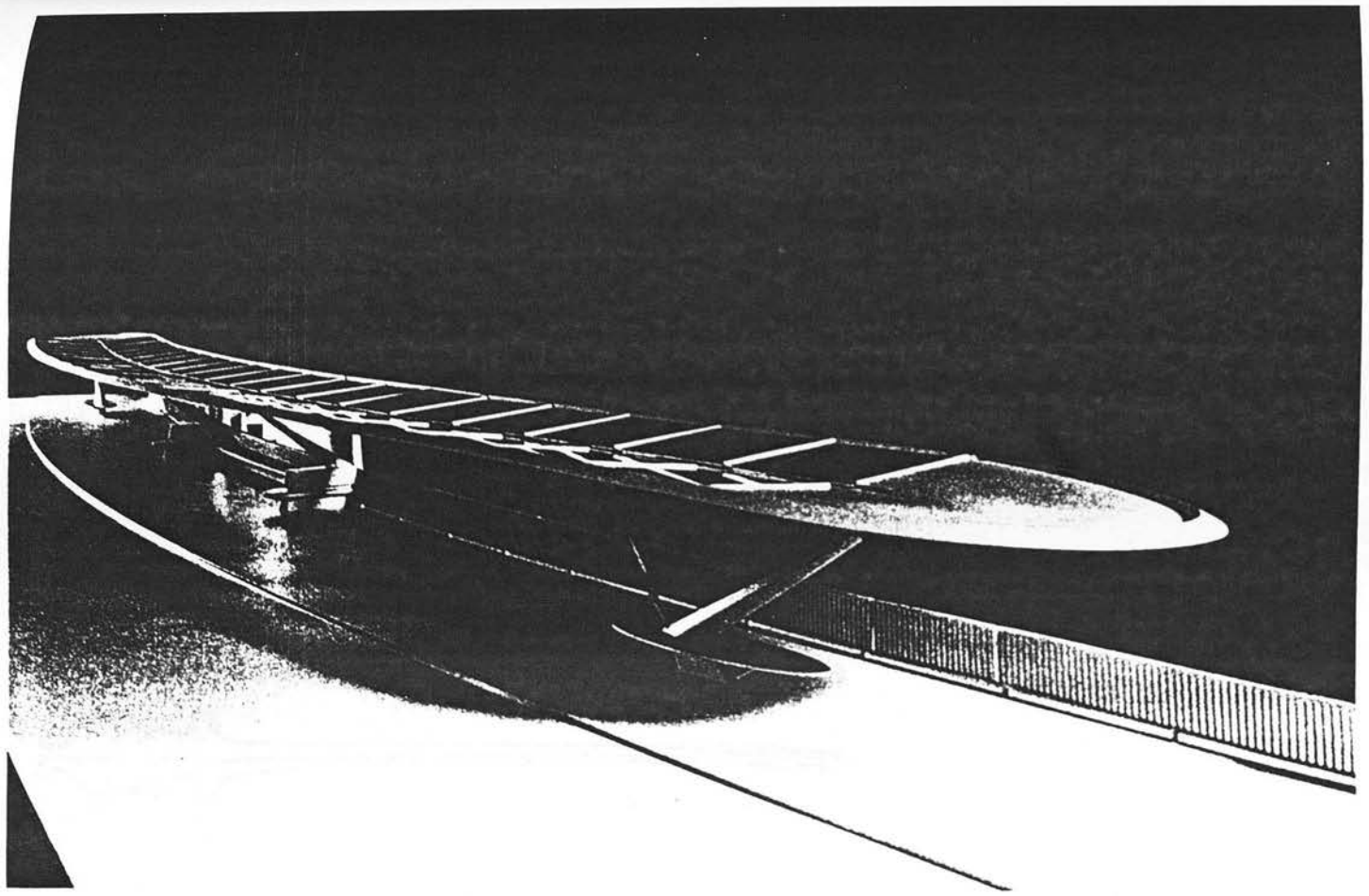


Planta de cabaret
Cabaret floor plan

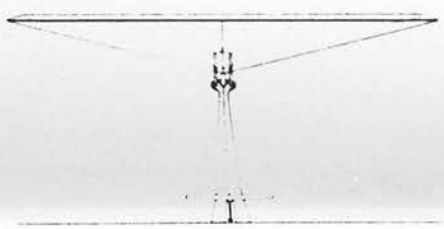
Plantas y secciones de la escalera



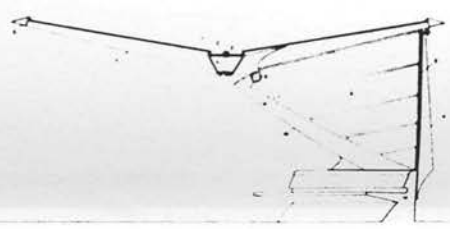
Plans and sections of the staircase



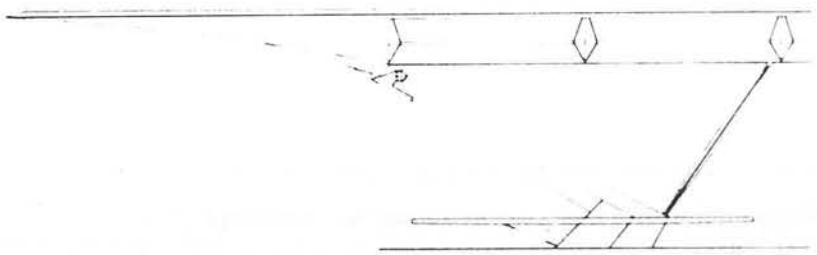
1 Vidrios de seguridad.
2 Goterón.
3 Sumidero.
4 Estructura longitudinal.
5 Luminaria (fluorescente).
6 Estructura de borde.
7 Estructura transversal.
8 Articulación.
9 Acristalamiento lateral.
10 Soporte oblicuo.
11 Costilla lateral.
12 Banco corrido.



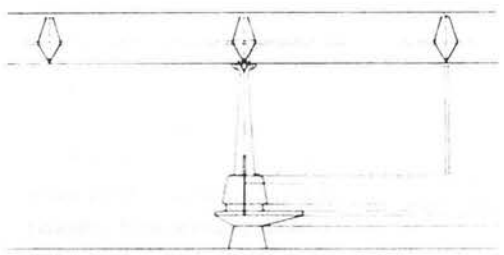
Alzado lateral
Side elevation



Sección transversal por banco
Cross section through bench

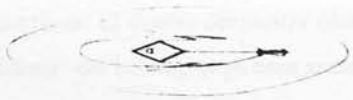


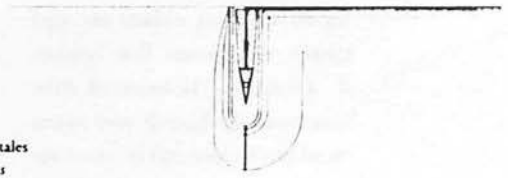
Alzado frontal. Detalle
Front elevation. Detail



Alzado frontal. Detalle de banco
Front elevation. Detail of bench

1 Safety glassing.
2 Throating.
3 Drain.
4 Longitudinal structure.
5 Fluorescent lights.
6 Edge structure.
7 Transverse structure.
8 Articulation.
9 Side glassing in.
10 Oblique support.
11 Side rib.
12 Continuous bench.

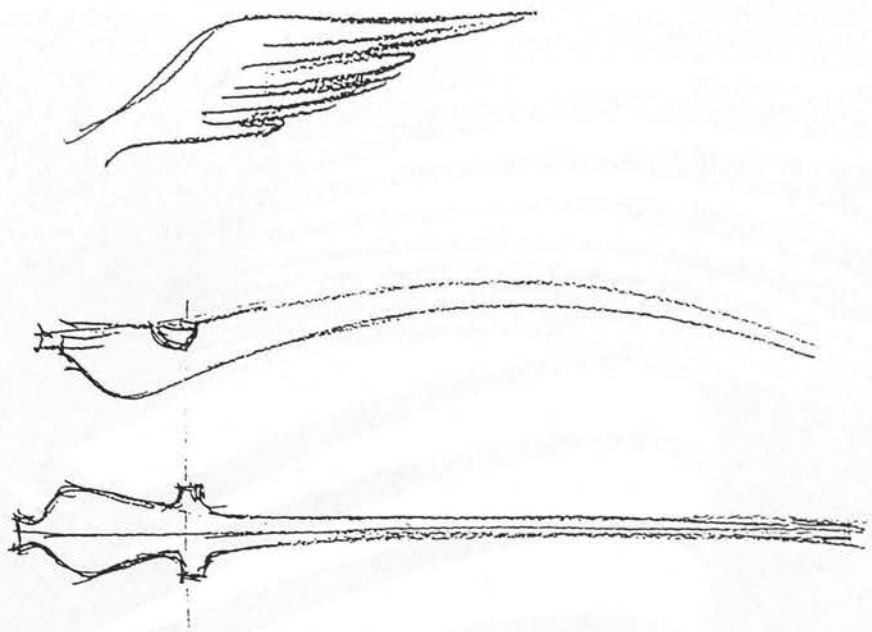


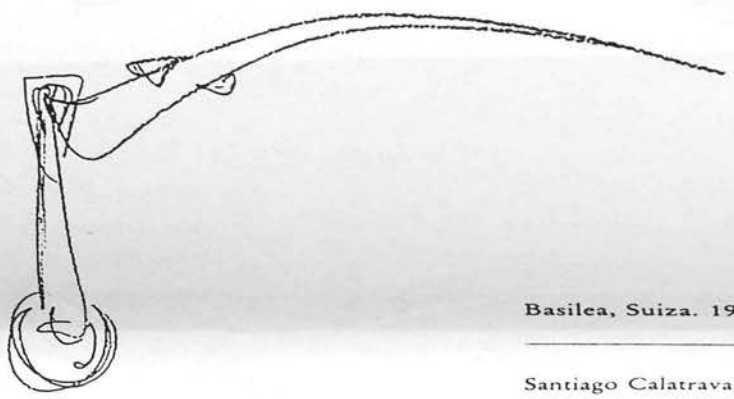


Secciones horizontales
Horizontal sections

SZI Basel

Pabellón Anual del Cemento / Annual Cement Pavilion



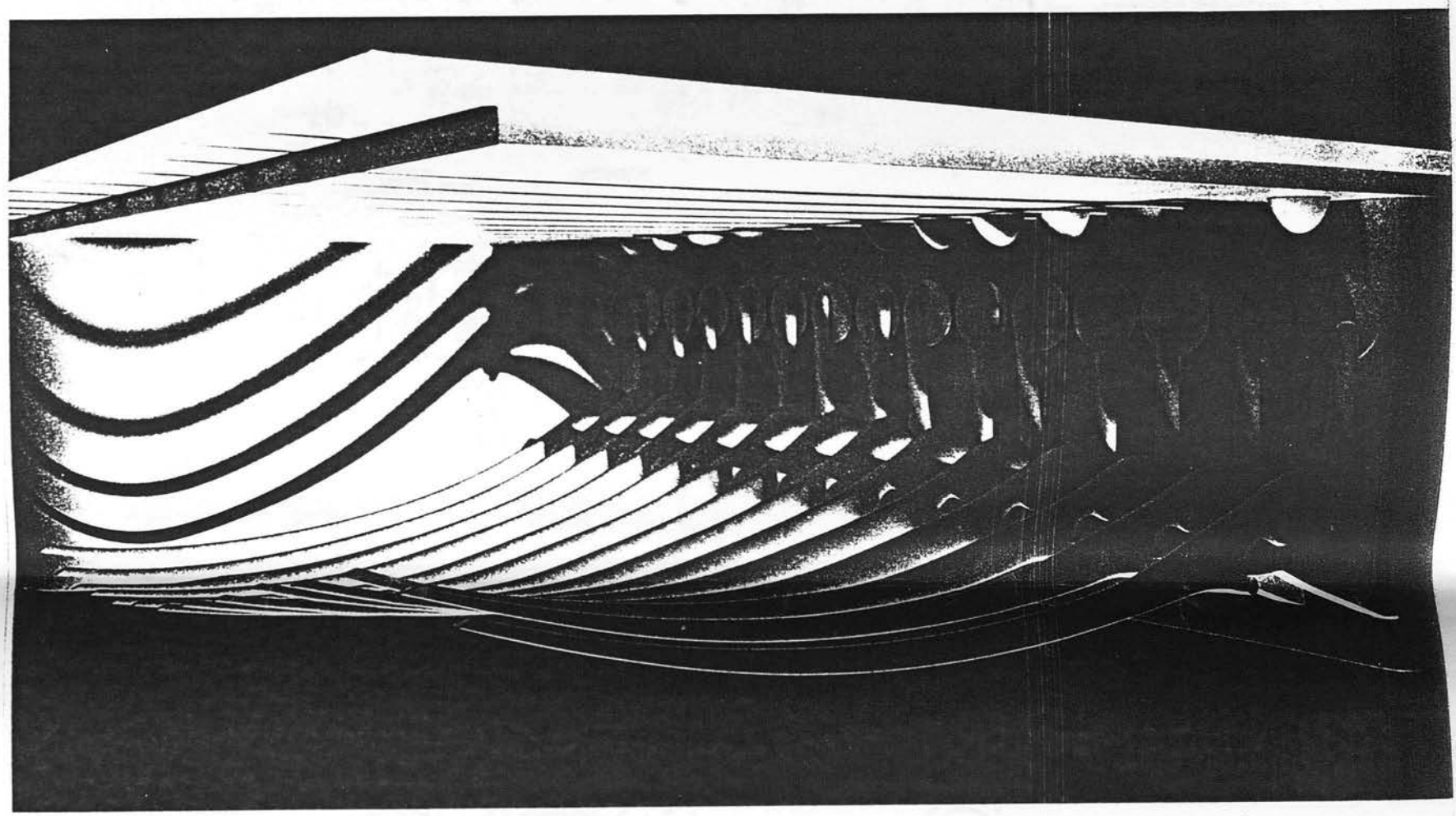


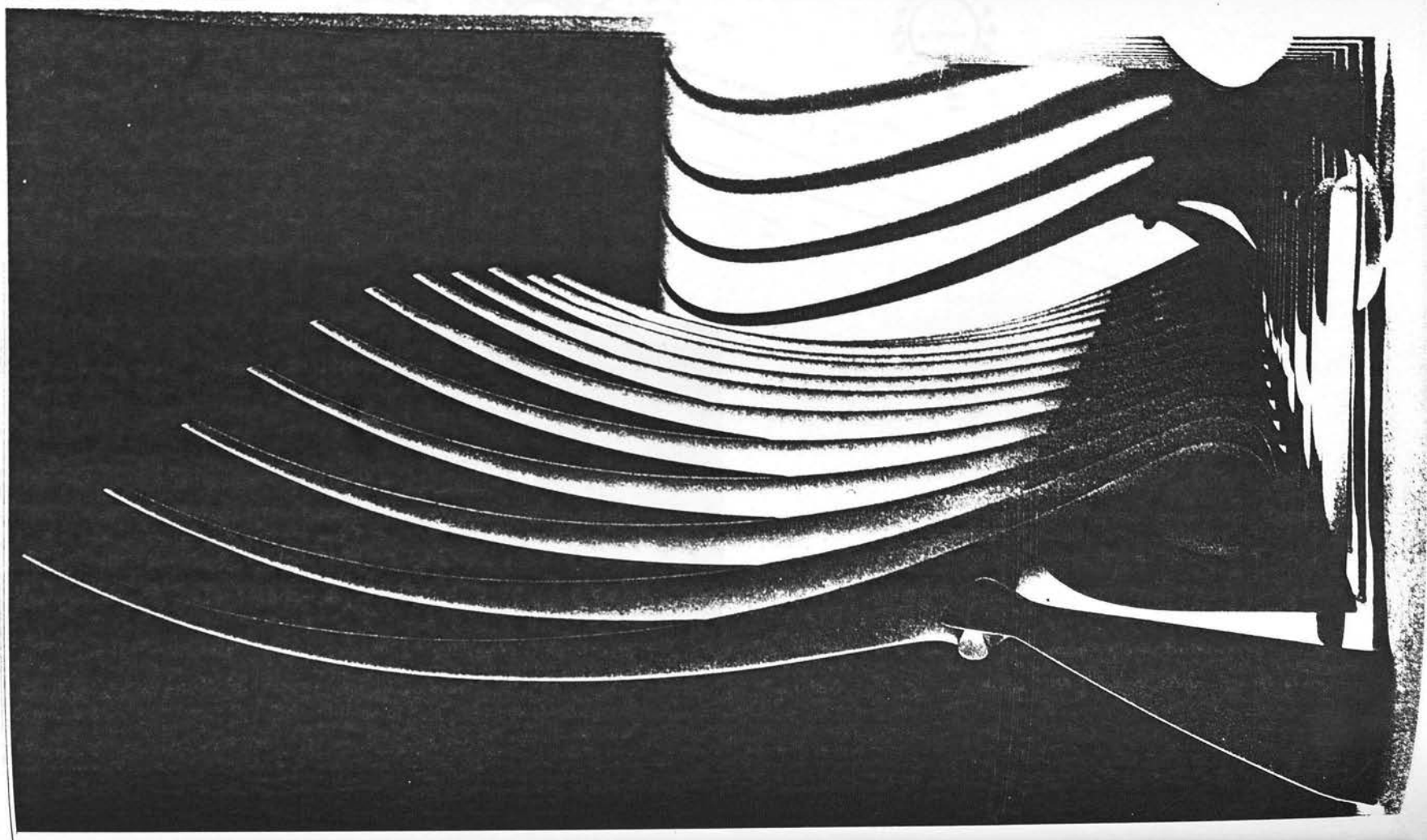
Basilea, Suiza. 1989

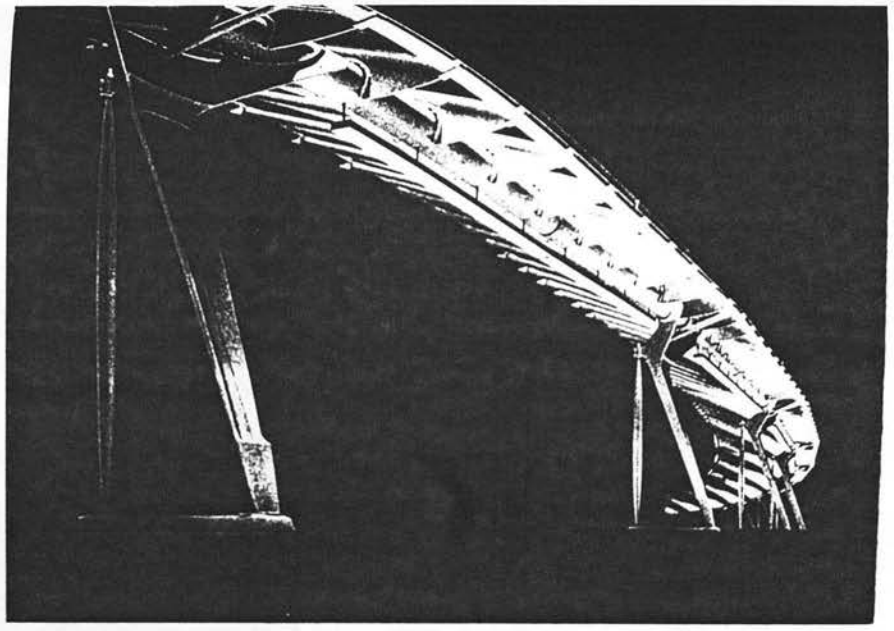
Santiago Calatrava

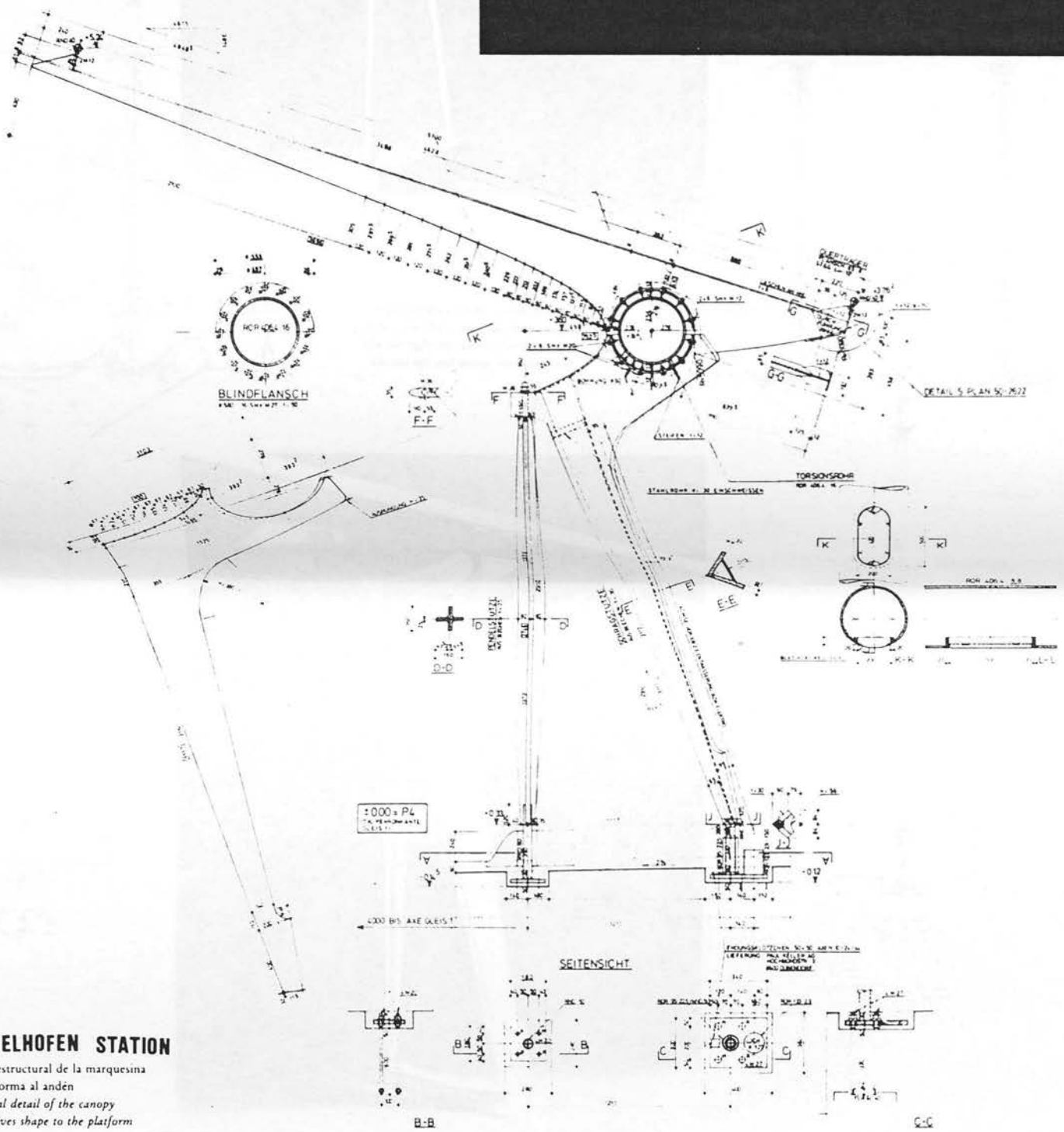
Es esta una construcción provisional en la que se pretenden demostrar las múltiples posibilidades del cemento prefabricado. Conceptualmente, se trata de una máquina de hormigón, capaz de cambiar su forma a través de los movimientos coordinados de sus elementos. Como resultado de la modulación de la luz, el juego de sombras sobre el pavimento cambiará, con una periodicidad armónica, de manera continua. El diseño demuestra cómo, a través del movimiento, las formas del hormigón pueden entroncarse con las de la naturaleza.

This is a provisional construction in which we intend to show the extreme posibilities of pre-fabricated concrete. Conceptually, there is a machine of concrete, able to change its own shape through the coordinate movements of its elements. Through the modulation of the light, the shadow pattern in the pavement will continuosly change with harmonical periodicity. It shows how through the movement the forms of the concrete can be related to the forms of nature.





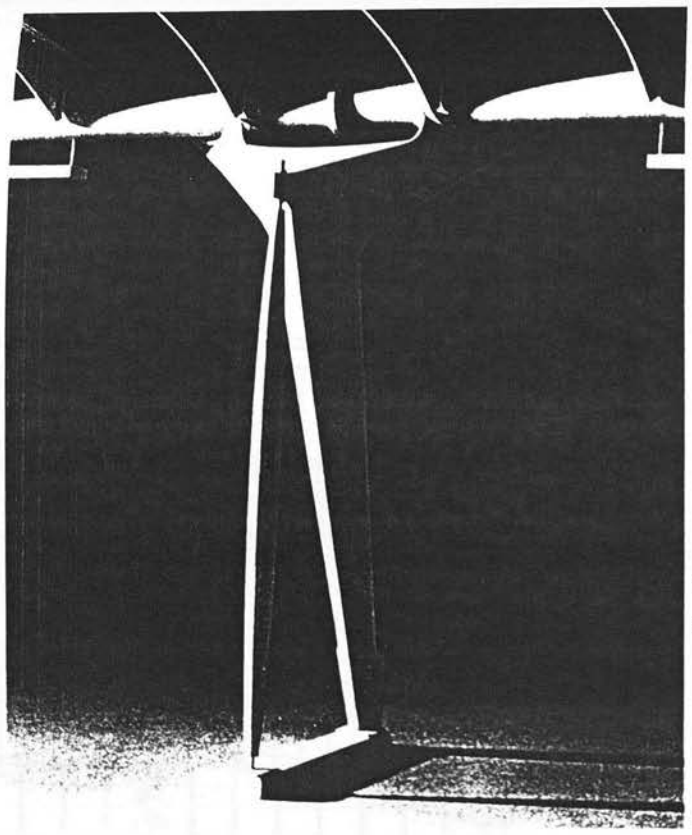


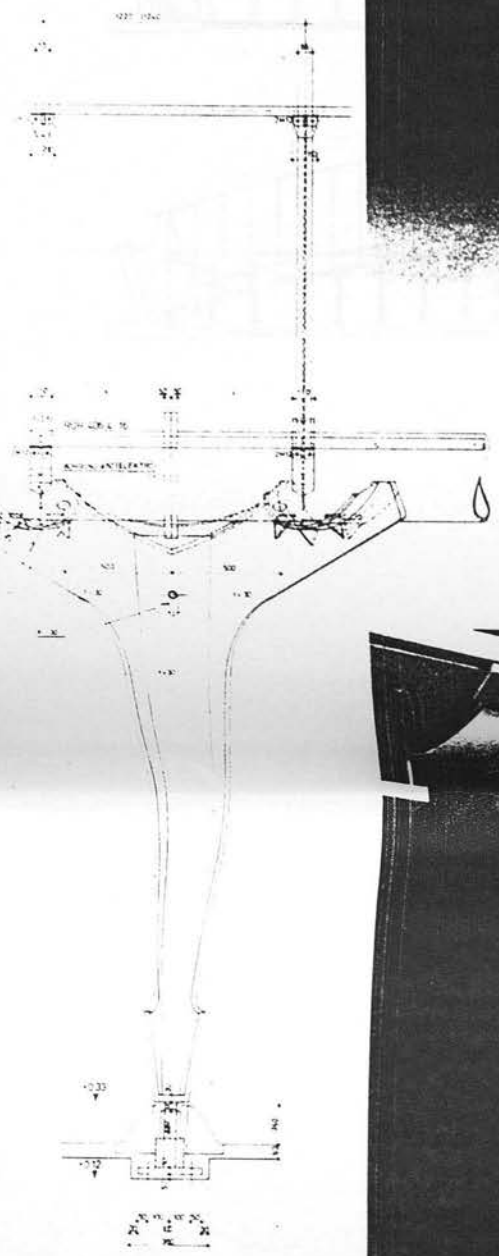


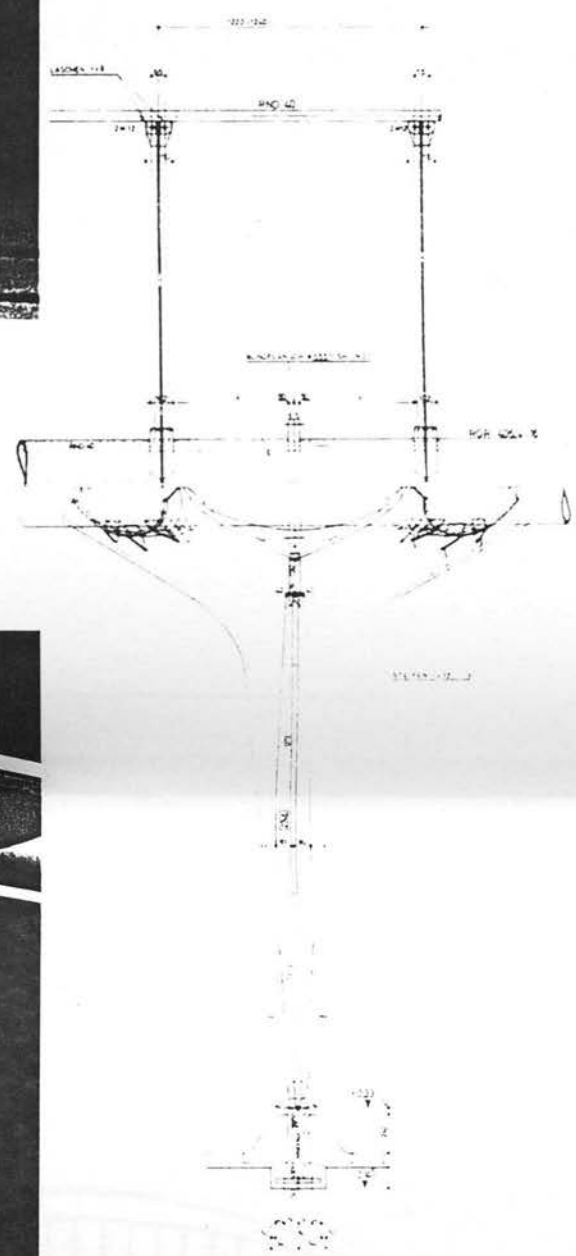
STADELHOFEN STATION

Detalle estructural de la marquesina que da forma al andén

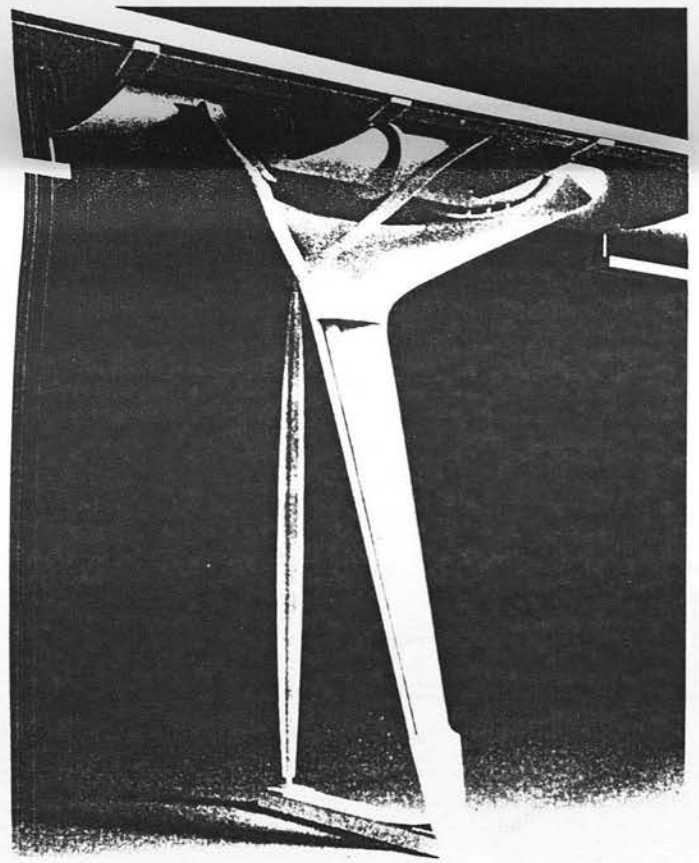
Structural detail of the canopy which gives shape to the platform



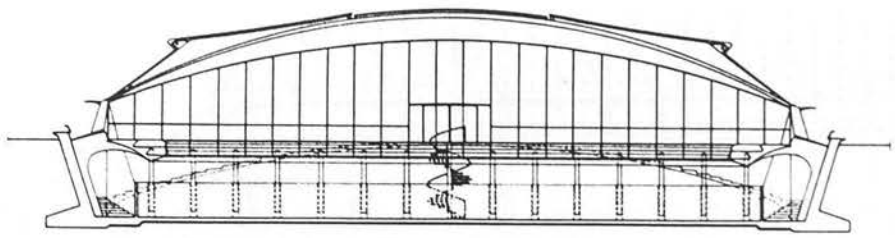




A la derecha y arriba: alzado frontal
A la izquierda y abajo: alzado posterior
On the right and above: front elevation
On the left and below: rear elevation



Centro Deportivo / Sports Center



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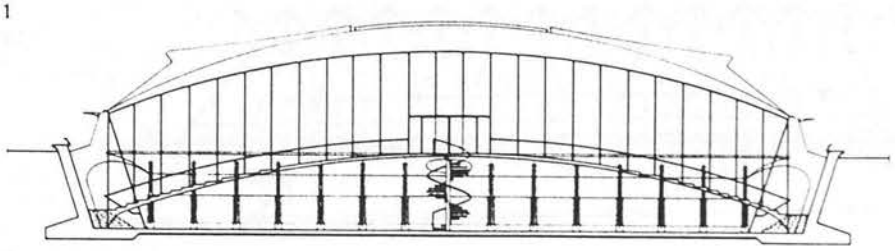
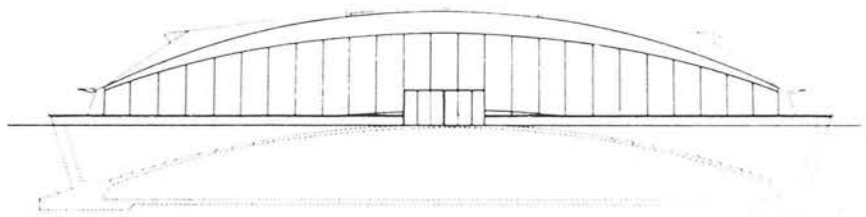


Ginebra. Suiza, 1989

Santiago Calatrava



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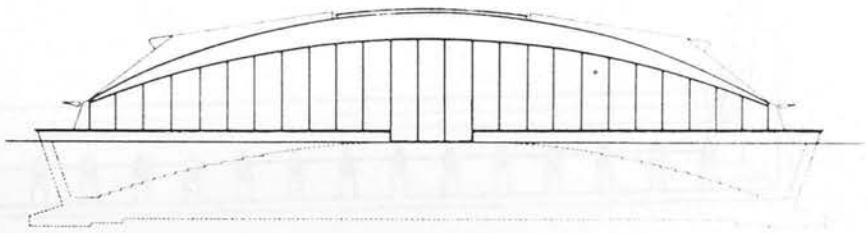
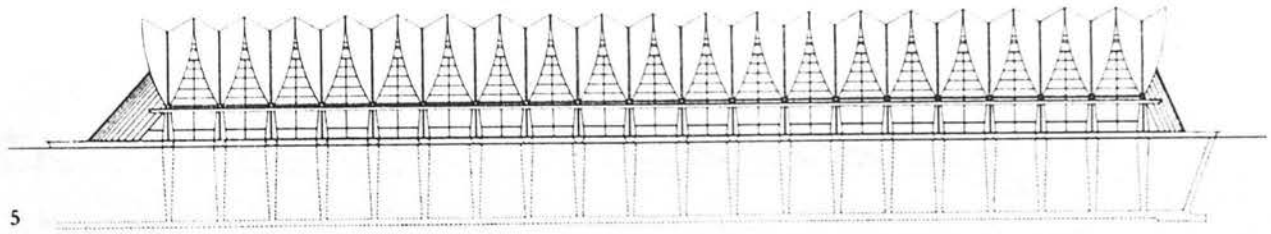


3

1 Secciones horizontales / *Cross sections*
2 Alzado Noroeste / *North West elevation*
3 Alzado Sureste / *South East elevation*
4 Planta / *Plan*
5 Alzado Suroeste / *South West elevation*



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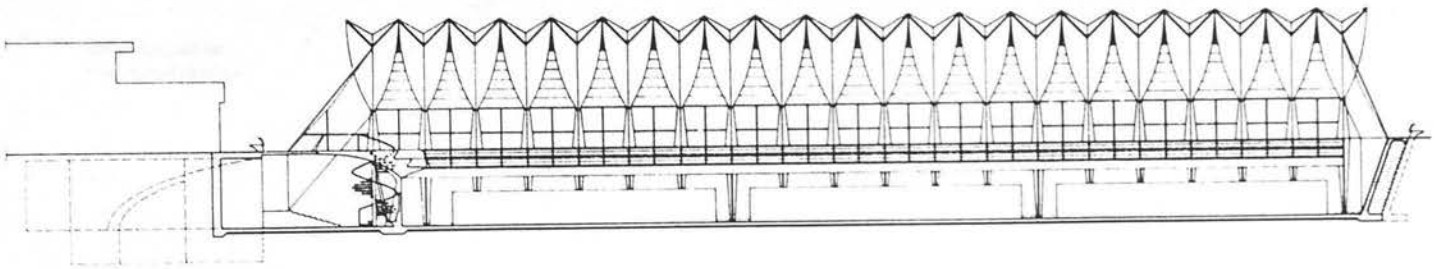

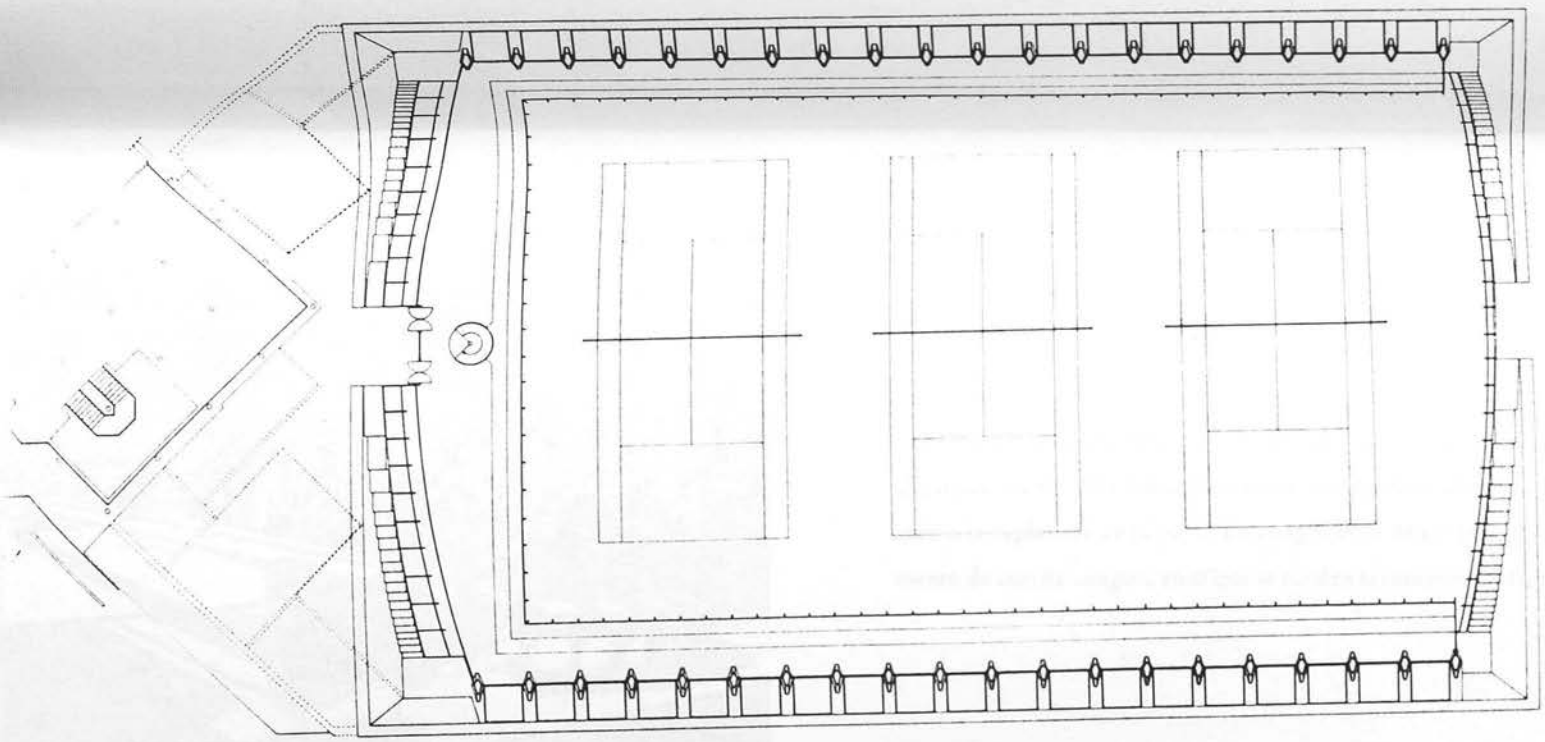

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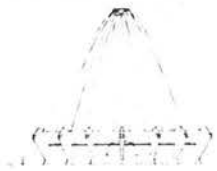
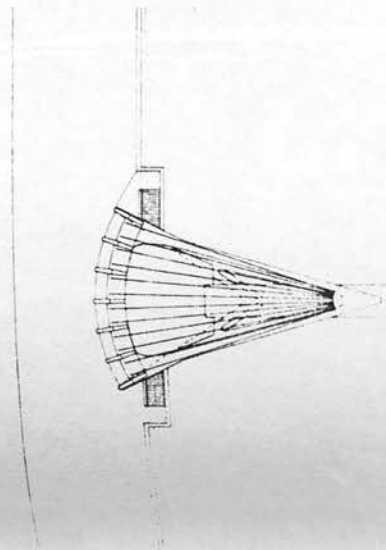


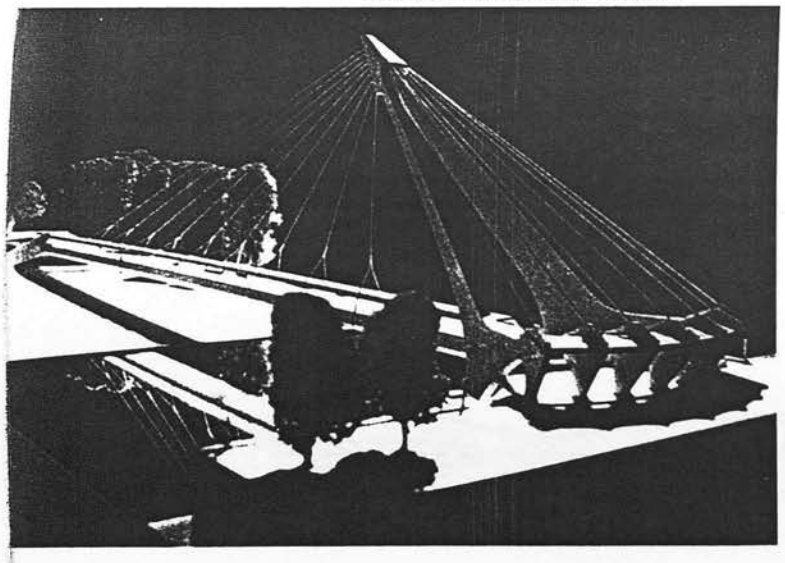
Alzado frontal
Front elevation

Alzado longitudinal
Longitudinal elevation



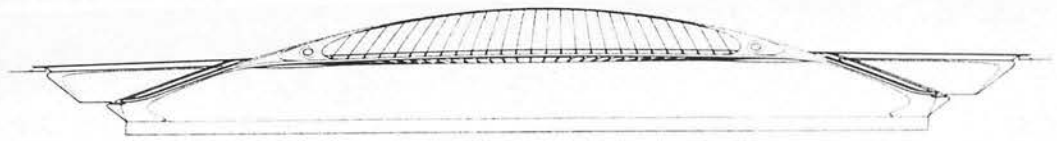
Planta
Plan

SEGRE PEDESTRIAN BRIDGE

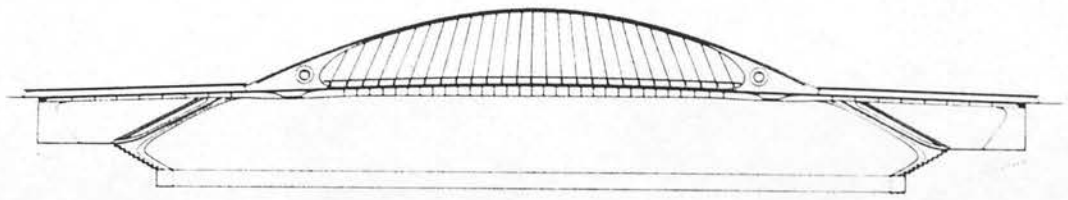


El puente constituye un gran pórtico de entrada, un acceso en el que las tensiones creadas y desarrolladas en el entorno urbano pueden ser fácilmente liberadas. Su pasarela, con cubierta de aluminio, se presenta como un tunel con ventanas horizontales en toda su extensión, que desemboca en dos grandes rampas que conducen al parque situado más abajo. Una amplia escalinata proporciona acceso a la explanada de la torre. La imagen es la de un gran instrumento de cuerda antiguo, en el que se funden la función y la forma.

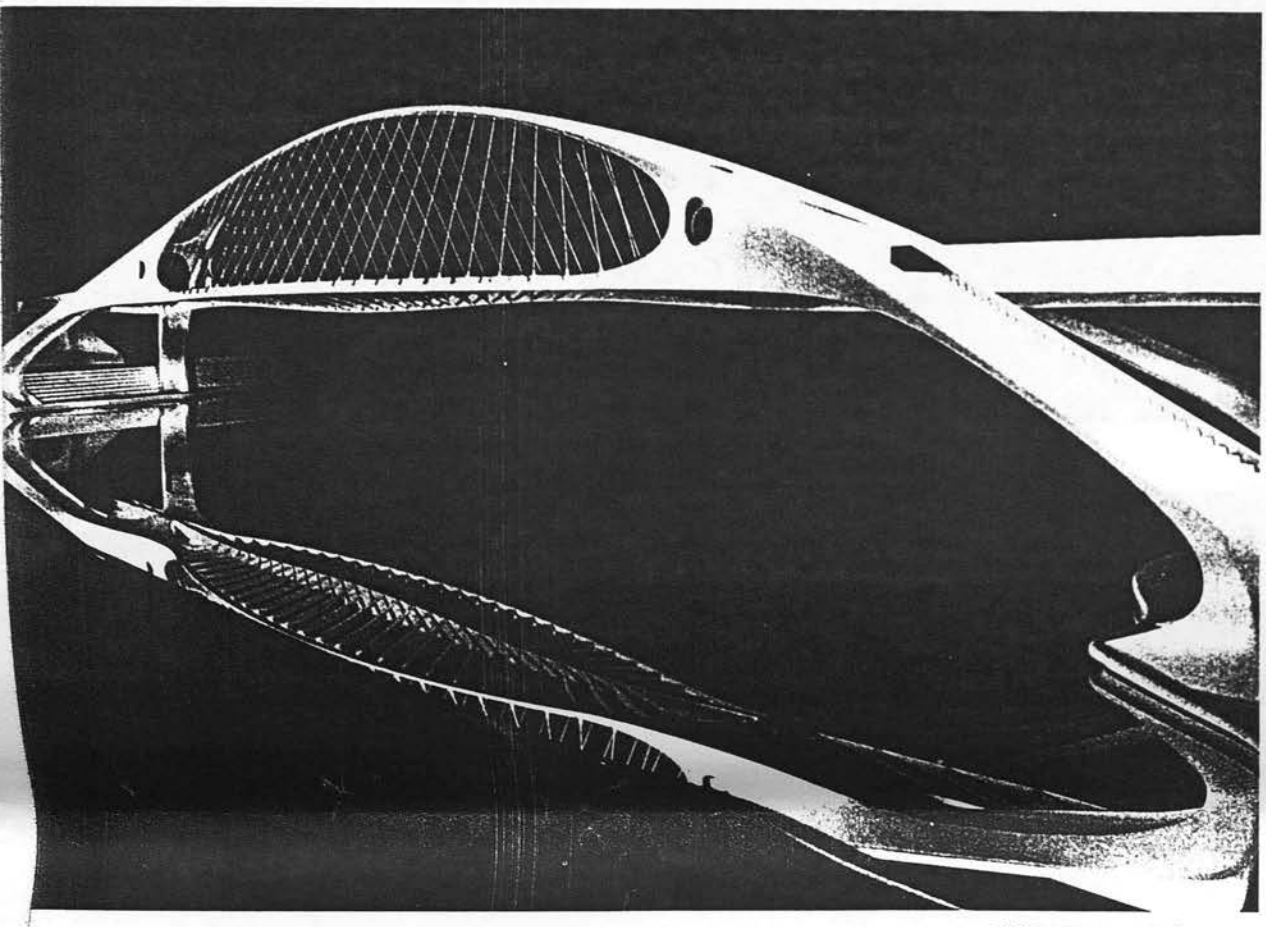
Fotografías:
Heinrich Helfenstein



Alzado / *Elevation*

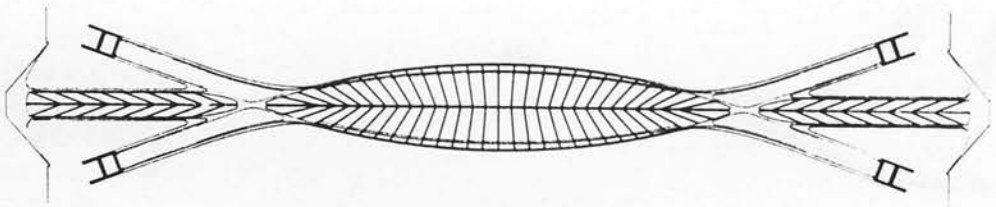


Sección longitudinal
Longitudinal section



LUNGARNO BRIDGE

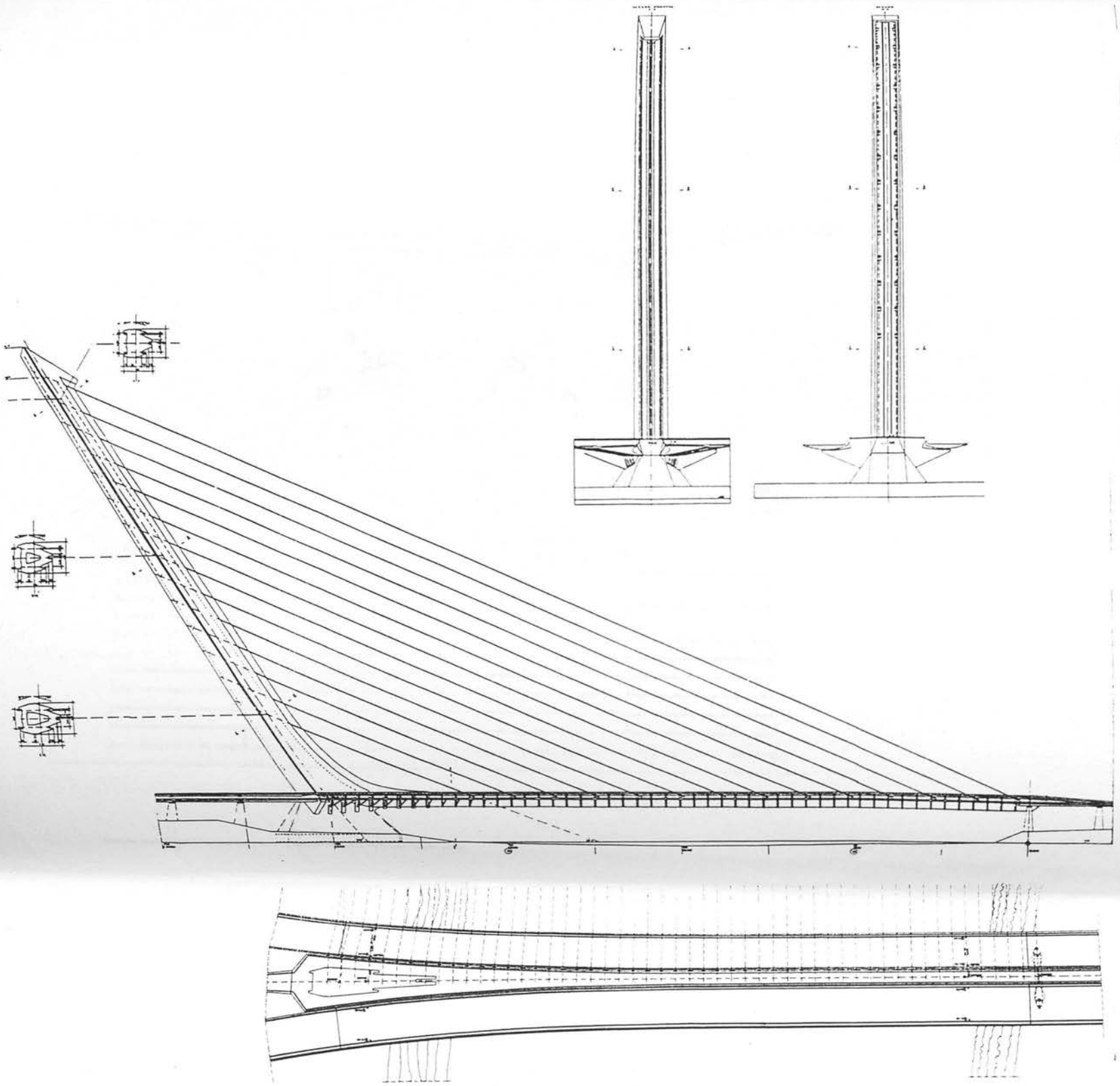
Planta / *Plan*



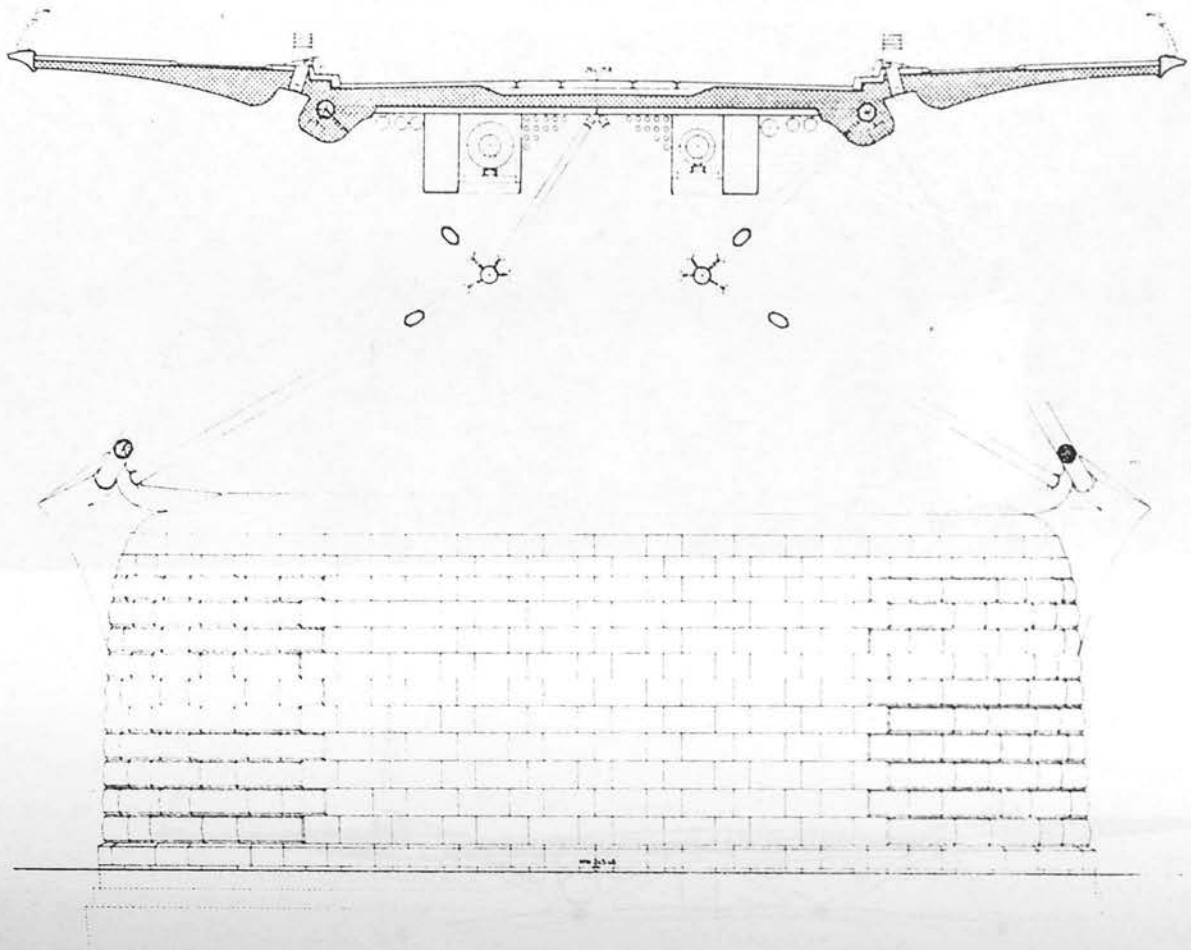
Intradós / *Inside surface*

SEVILLA BRIDGE

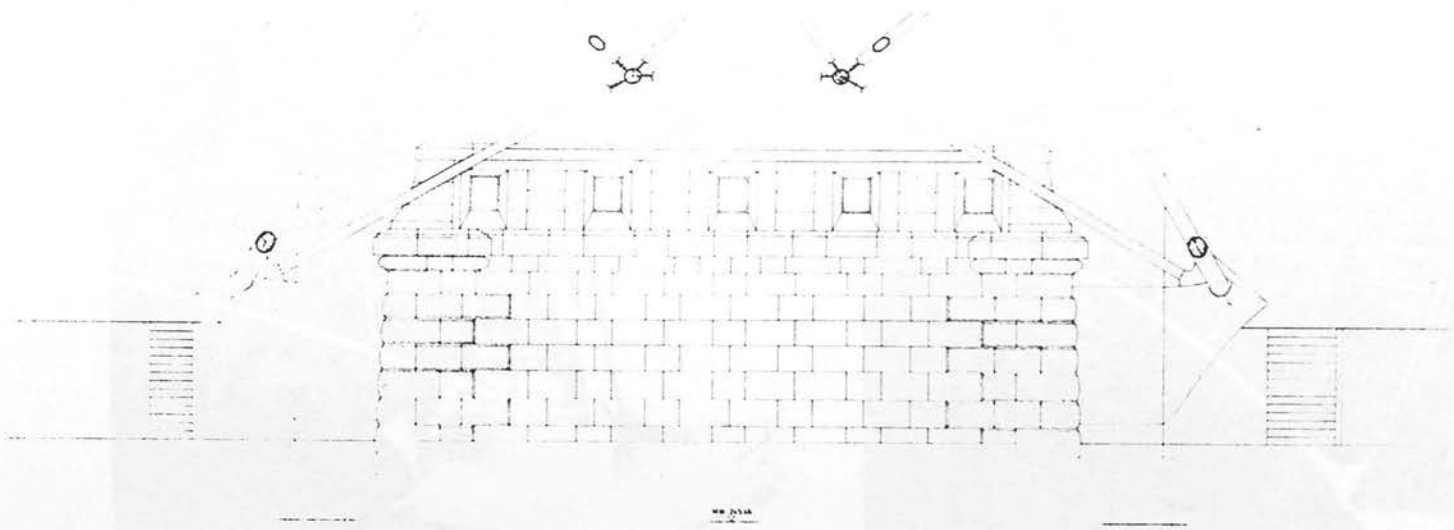
The two twin structures of cable-stayed bridges with towers at one side, a span of 250 metres and a height of 162 metres, were conceived as boundary constructions for the entire exhibition site.



En la página de la izquierda:
Planta de situación y alzado general
En esta página:
Planta y alzados de uno de los tramos colgados
On the left page:
Site plan and overall elevation
On this page:
Plan and elevations of one of the suspended parts

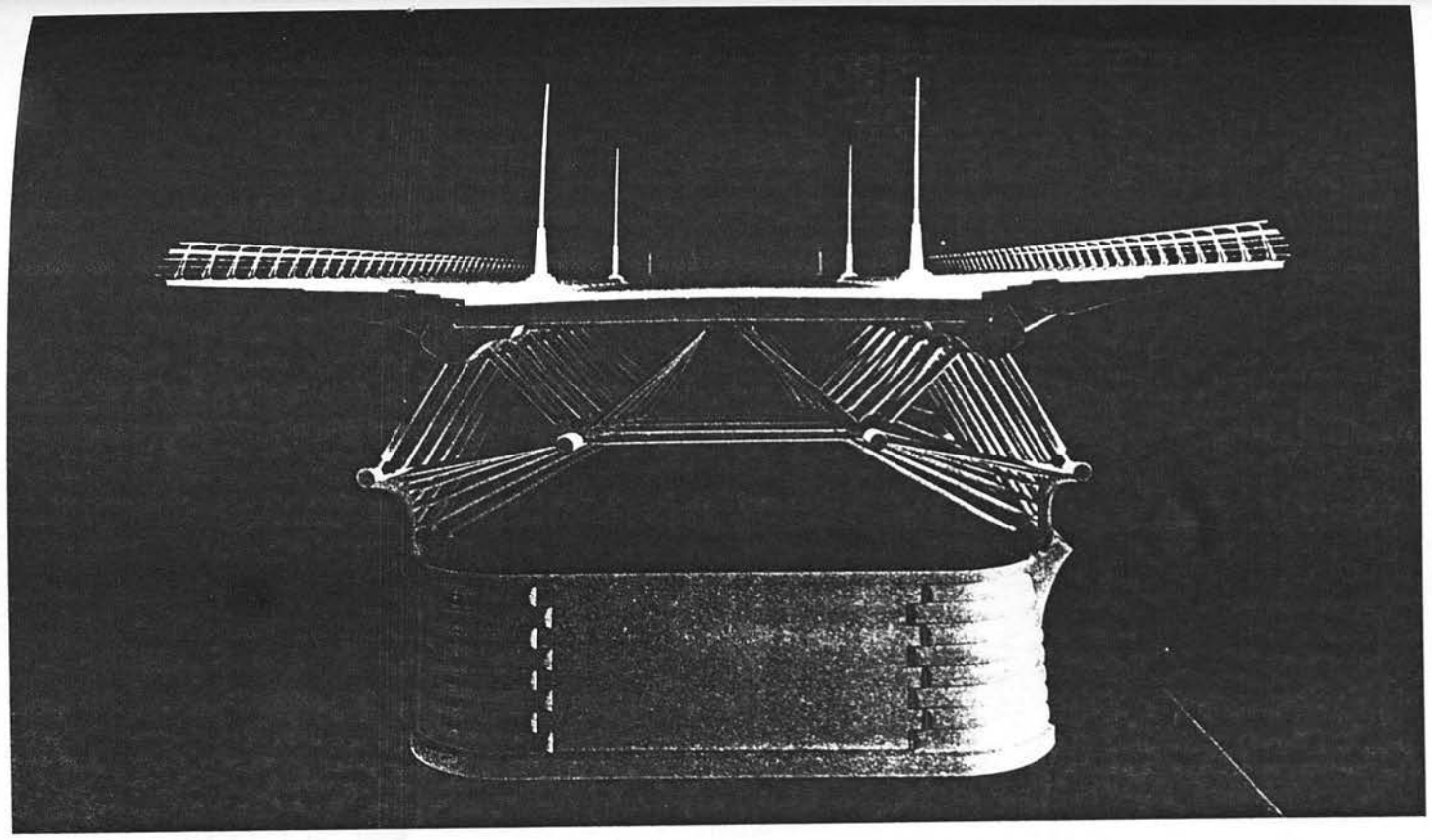


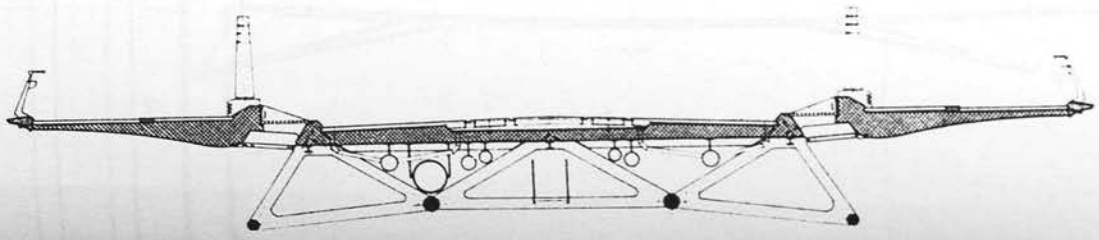
Sección transversal y alzado del pilar / *Cross section and elevation of the pillar*



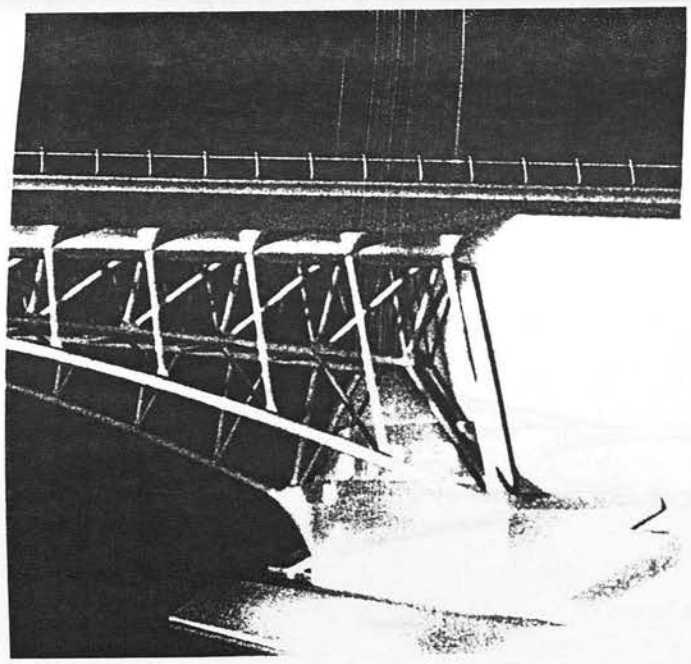
Alzado del soporte de arranque de la estructura / *Elevation of one end of the structure*

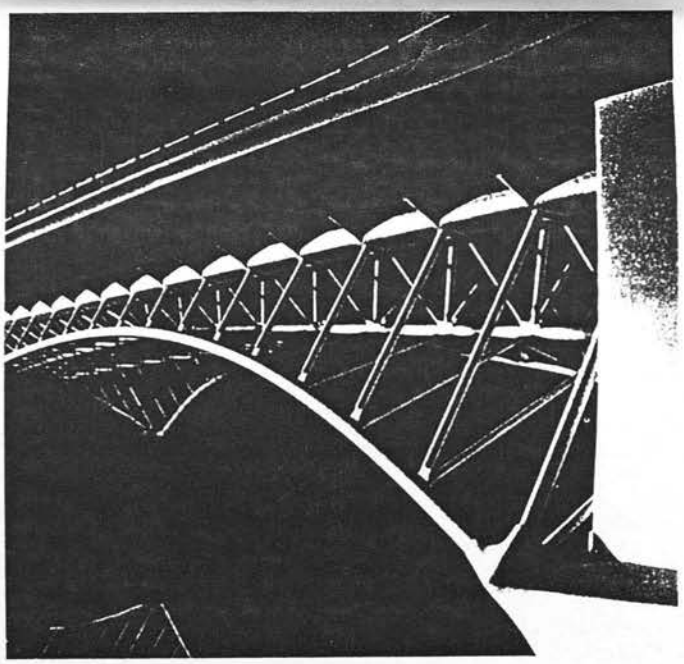
RHIN BRIDGE



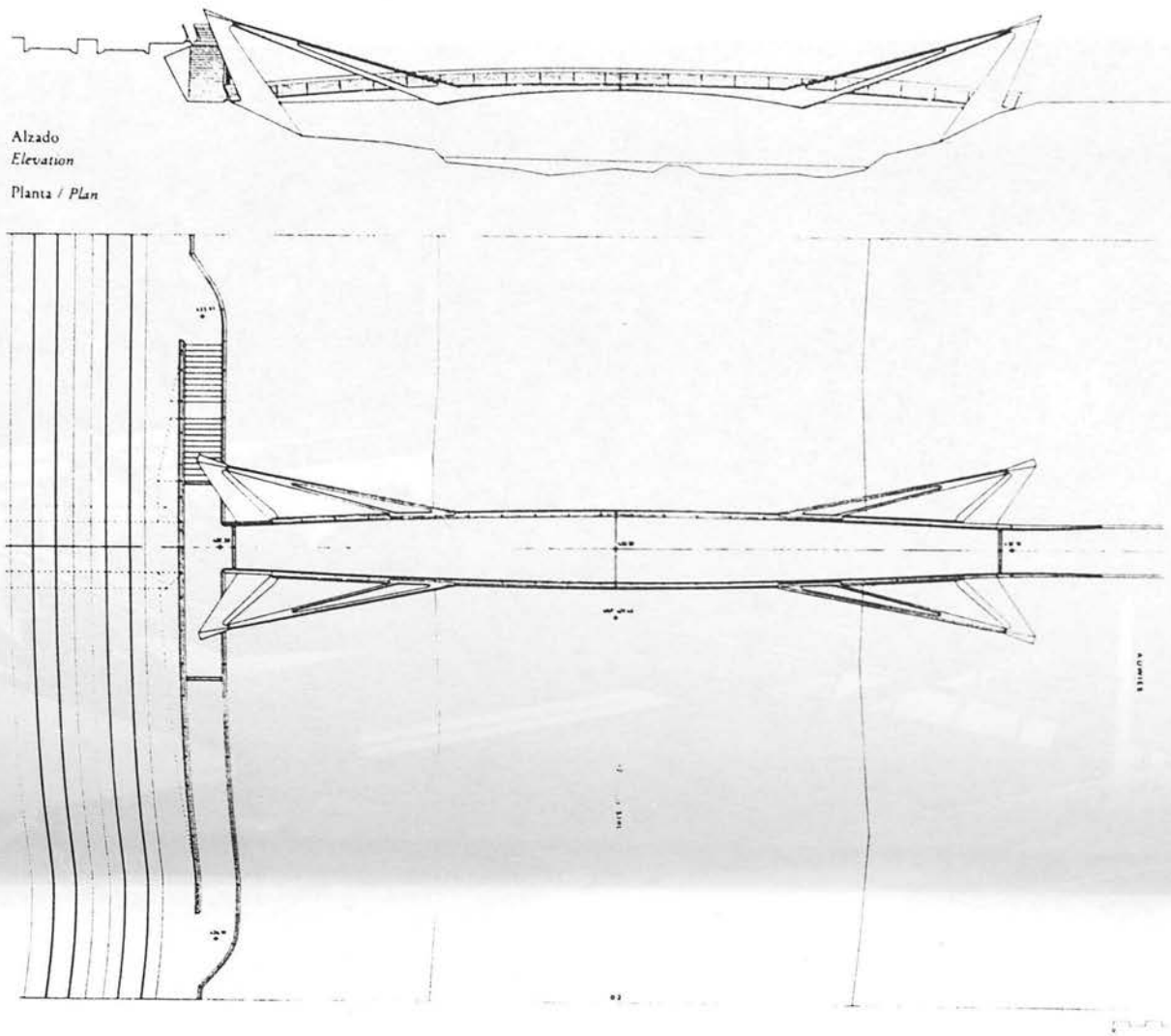


Sección transversal (anteproyecto) / *Cross section (preliminary project)*



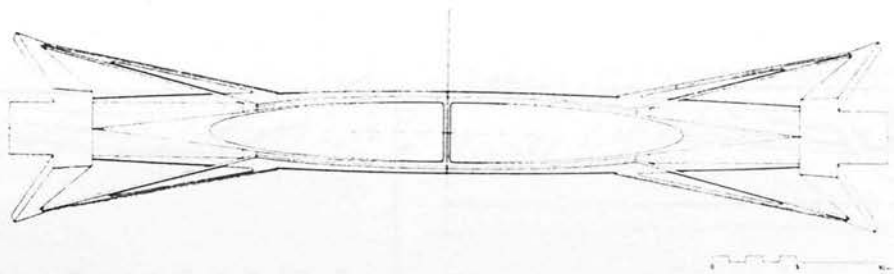


Puente peatonal sobre el Sihl / Pedestrian bridge over the River Sihl Zurich, Suiza. 1988

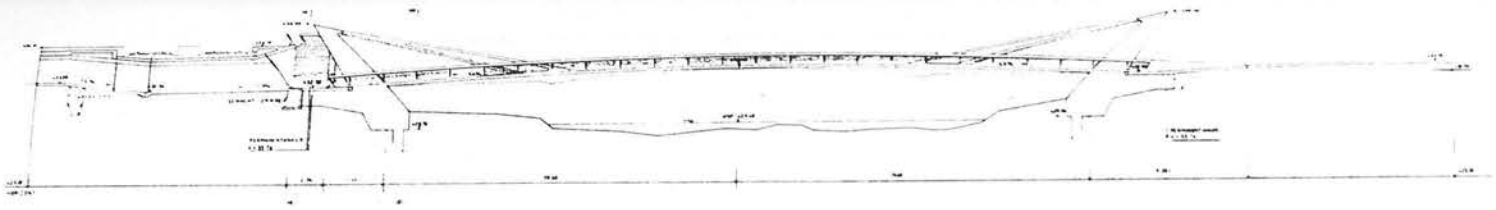


Alzado
Elevation

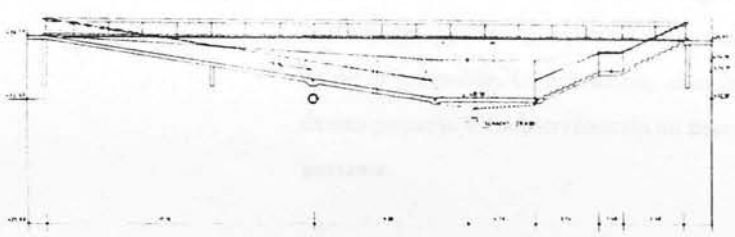
Planta / *Plan*



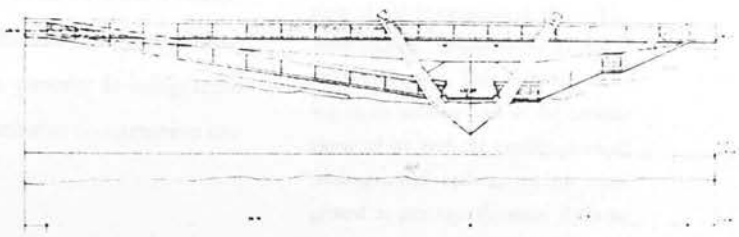
Intradós / *Inside surface*



Sección longitudinal / *Longitudinal section*

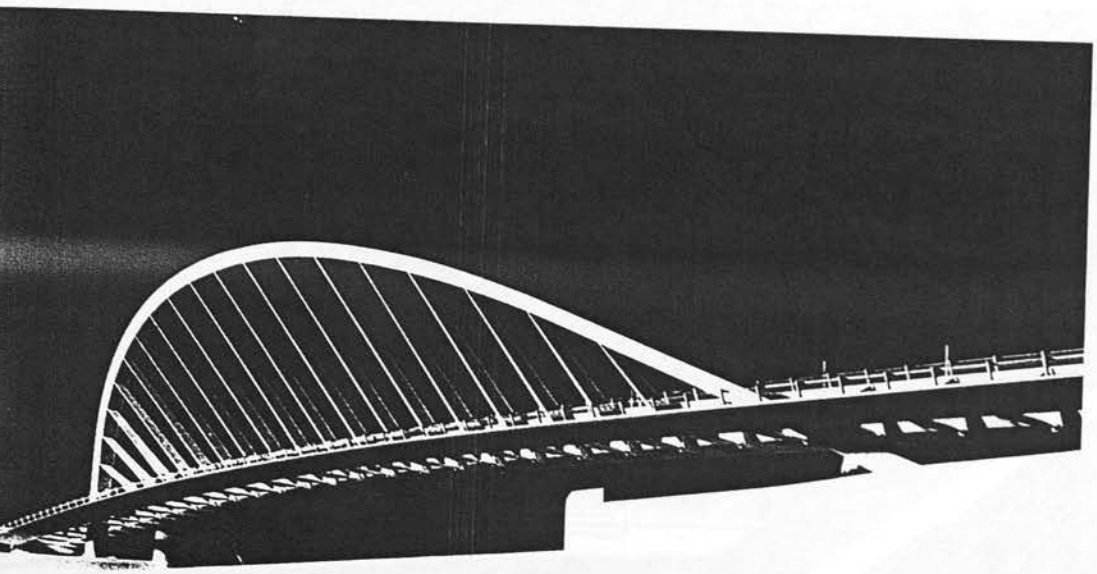


Sección transversal AA / *Cross section AA*



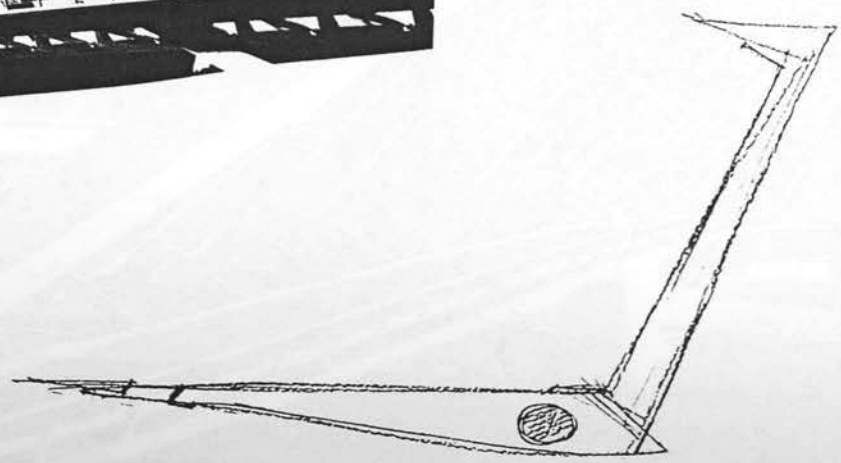
Sección transversal BB / *Cross section BB*

Puente sobre el Sena / Bridge over the River Sena



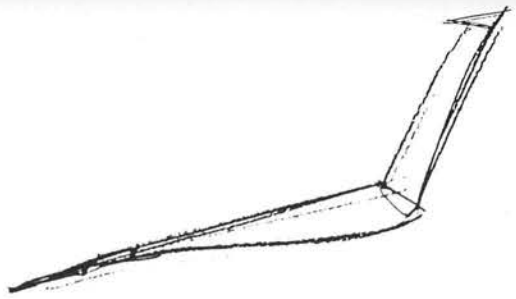
París, Francia. 1987-88

Santiago Calatrava



El proyecto Pont Gentil consiste en un puente urbano que enlaza los distritos 12 y 13 del Gare de Lyon con el Gare d'Austerlitz. La unilateralidad de su alineación espacial subraya la demarcación existente entre el viejo y el moderno París. El rasgo más sobresaliente del puente es su asimetría, lo que resulta sumamente sorprendente en su sección y en la sesgadura de su arco. Produce el efecto de algo nuevo, extraordinario, bello, técnicamente al borde de lo imposible. La estructura, además, permite la integración de una pequeña vía subterránea sin un desembolso constructivo importante.

The Pont Gentil project represents an inner-city bridge linking the 12th and the 13th arrondissements or the Gare de Lyon with the Gare d'Austerlitz. At the same time, the unilaterality of its spatial alignment underlines the demarcation of old from modern Paris. The outstanding feature of the bridge is its asymmetry. This is most striking in its section and in the unique skew of its arch. It enables a small underground railway to be integrated as per specification without a major constructional outlay.



AUSTERLITZ
Fotografías y fotomontaje
Photographs and montage
Heinrich Helfenstein

CASE STUDY 31: KANSAI AIRPORT

(Extracted from the Renzo Piano Building Workshop, RIBA 1989)

Architect: Renzo Piano

1 Architect's Concept

The reality of runways, air planes, technology and science on one side will live harmoniously with an invasion of nature and light on the other side. The terminal itself is where the transition occurs. Inside it one will find two "valleys" of daylight and penetrated by nature, one on the land side and the other on the air side of the station. Within the protected environment of the building, these "valleys" echo external nature, and become an instinctive visual cue for passengers moving across the building. Movement through the terminal is intended to be simple and direct.

A sense of the structure of the main hall at level three (which is visible from the first and second levels via the open landscaped spaces), will accompany the traveller walking from land to air and back. Piano wrote: "I believe that structure, especially of an air terminal, should be the diagram of people moving through it. and all the atmospheric elements of the space - the light, the sound, the movement of the air - should contribute to the logic and intention of their movement."

The aerodynamically curved section of the roof is intended to encourage ventilation from the land side toward the air side of the building. Between the arches, the scoop-like form of the

roof assures laminar flow and separation between adjacent air zones. Maintenance is directly provided by rails and small trolleys at beam levels to give direct access to all roof equipment, lighting, smoke extraction and for cleaning the glass.

The structure of the arches themselves, with their longitudinal brackets, resembles aircraft fuselages, with the skin peeled off to let in light and to give a glimpse of the underlying construction. In the wings, at the entrance, light repetitive, braced arches - alternate arches strutted and supported - resemble early bi-planes, while the prestressed cable mullions and the suspended glass facade reflect the technology of today. The whole will be a structure at once sensitive and fragile, strong yet not overpowering, subtle and technically derived.

The architect intended this building to express a new balance between technology and nature, machine and man, the future and tradition.

2.0 Project: The Airport

2.1 Access

The land side access introduces travellers to the relationship between nature and the project. From the bridge the building is seen against the edge of the island developed as a wooded coastline. Cars loop through this landscape to reach the entrance canopy. The train station is sheltered by a glass roof surrounded by nature. The landscape penetrates the terminal in the form of valleys one crosses upon entering or leaving.

2.2 Zoning

The project follows the organisation described in the program. The levels and the main volume of the roof are connected physically and visually to each other by the landscaped valleys at either edge of the building and by a central atrium integrated with the "city" level. A subfloor has been inserted between the third and second levels. It houses offices and most concessions and duty-free areas, freeing up the domestic and international departure levels for rational circulation and unobstructed views. Moreover, the concentration of commercial and office activities within a dense nucleus limits the extent of potential fire hazards. The large spaces of the arrival and departure floors are thus safely connected. The floors are developed according to their separate purposes, giving to each an identifiable character. The domestic level and subfloor above it are treated as a miniature and rich city between the railway station and the boarding bridges. Above, the international departure level leads one to the air planes, and the landscape beyond. By contrast, the international arrival level, fringed by the bottom of the planted valley, draws one to the discovery of the nature of the island.

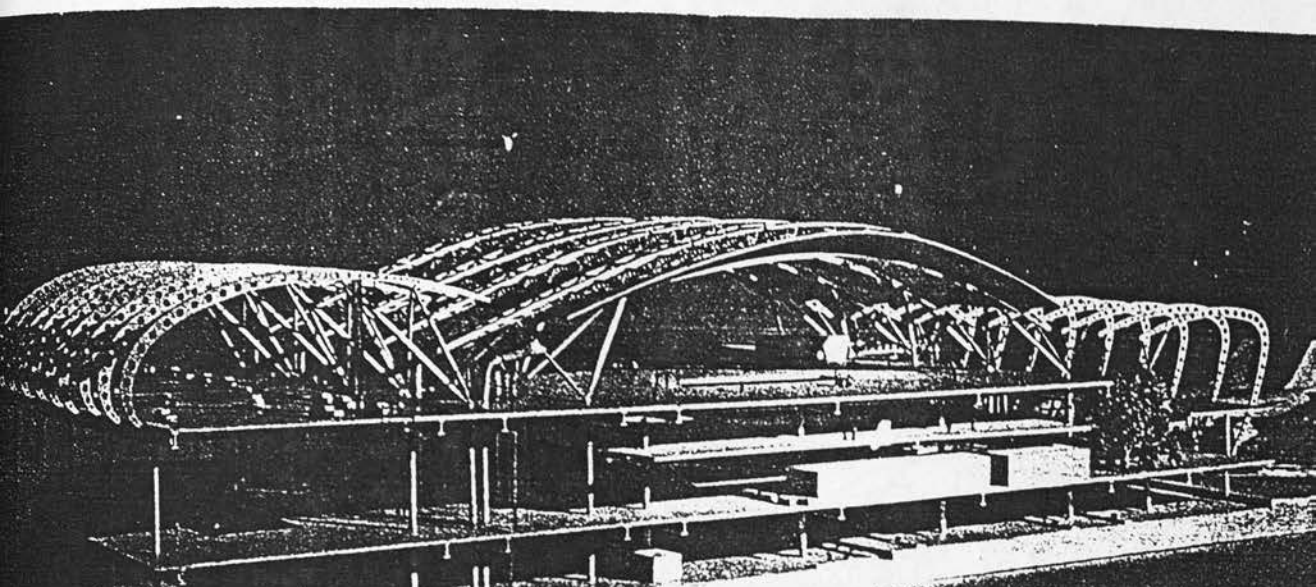
2.3 Circulation

In the terminal building counters and control points are organised to maintain a clear and direct circulation from landside to air side or back. The dynamics of space, structure and light support movement. Travellers are oriented by the clearly differentiated characters of land and air sides that they can see from their whole path. Vertical circulation takes place

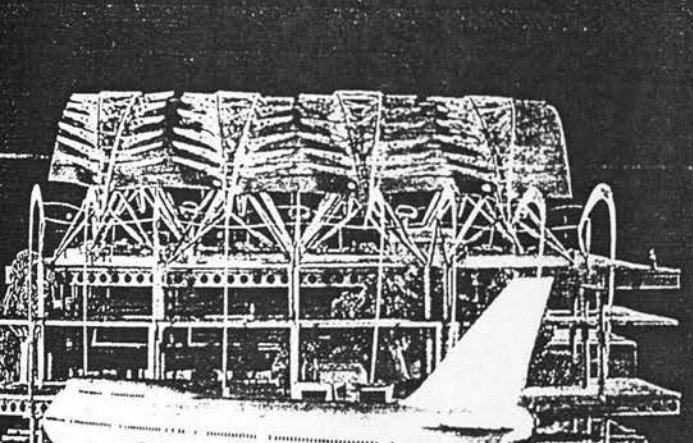
at the edges, in the valleys. A secondary circulation system connects the city level to the second and third floors. In the wings, departure and arrival flows are clearly separated. Nevertheless transparency is maintained and movement takes place between nature on one side and planes on the other, extending the references one orients himself with inside the terminal.

3.0 Structure and Form

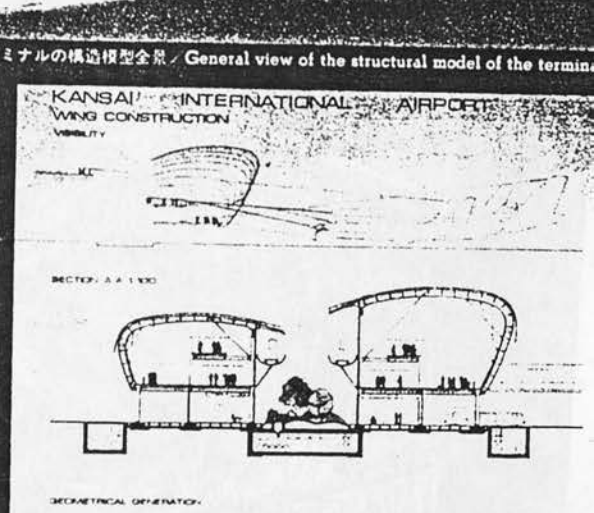
The exterior form is generated by a pure geometry based on rotated (wings) or translated (terminal) circular arcs, for rattional and economical construction, the wings rise to meet smoothly the ample undulation of the main roof creating a unique object with a clearly recognisable centre. The ceiling shells of the main space are profiled gently and fluidly to form open ducts for climate control. This lining for human comfort parts gently at the roof arches to let in light and reveal the intimate nature of the construction. The structure is articulated in elements of progressively finer dimensions relating to human perception and drawing on collectively held images of flight technology.



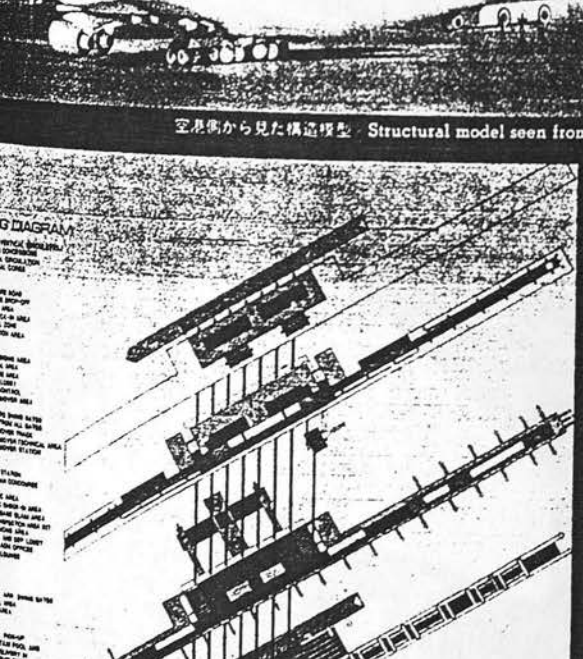
ターミナルの構造模型全景 / General view of the structural model of the termina

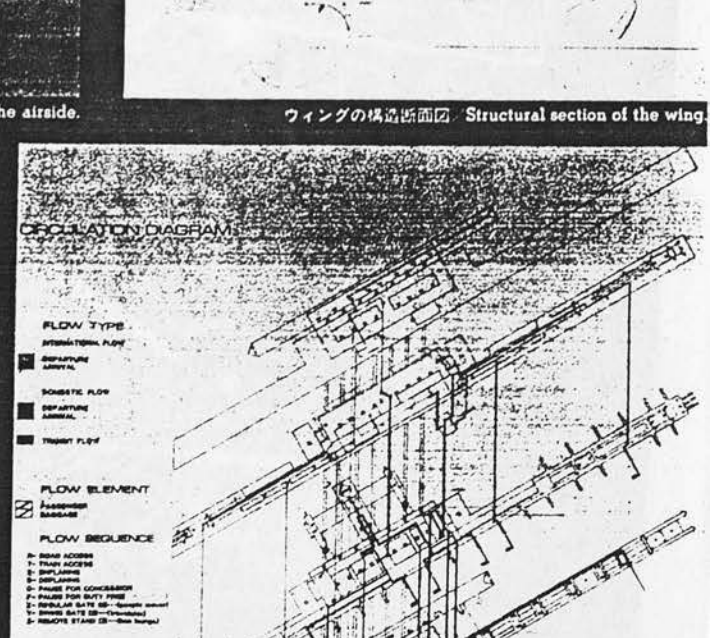


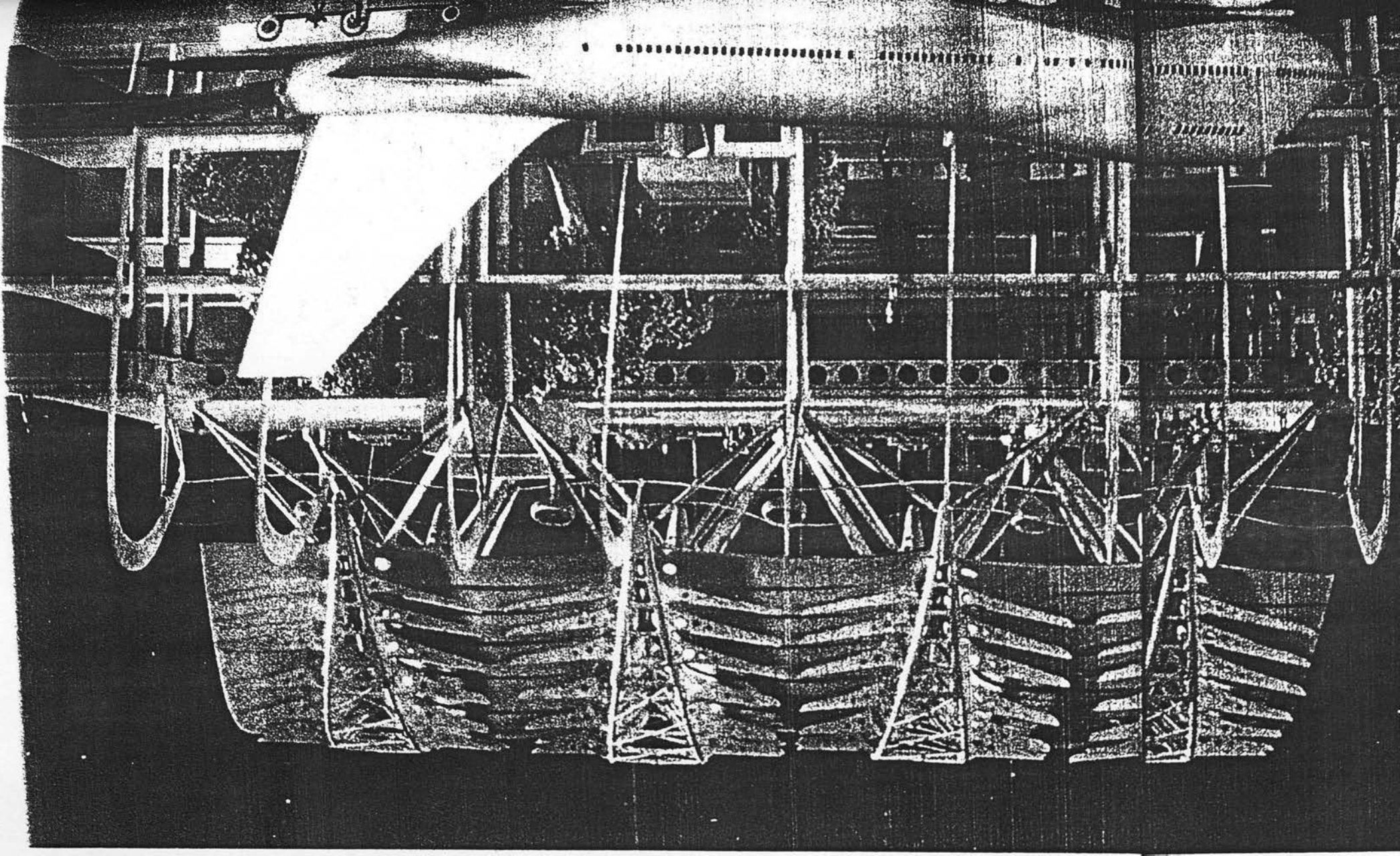
空港側から見た構造模型 / Structural model seen from the airside.



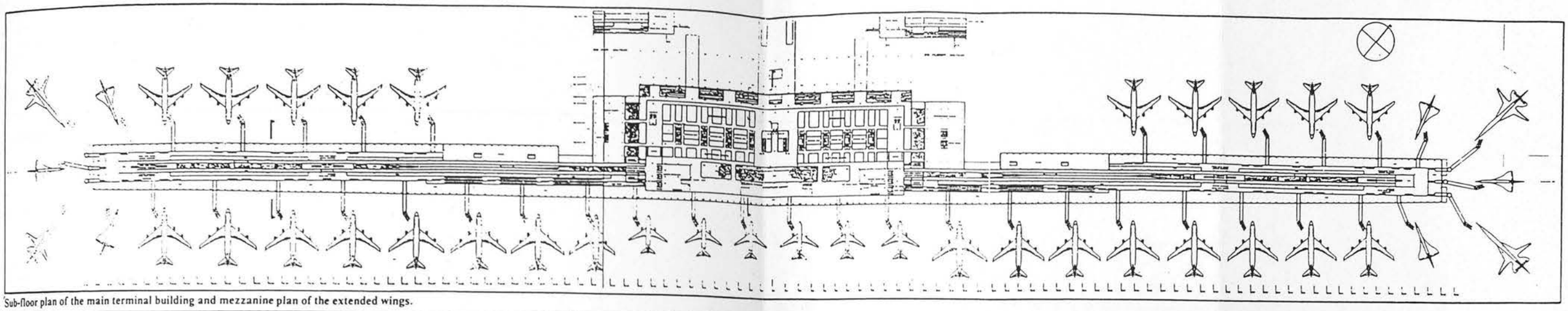
ウィングの構造断面図 / Structural section of the wing.



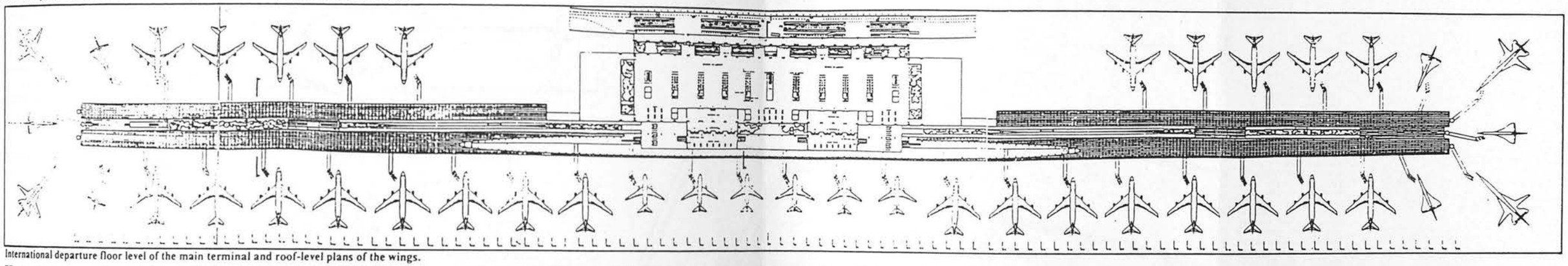




関西国際空港旅客ターミナルビル設計競技

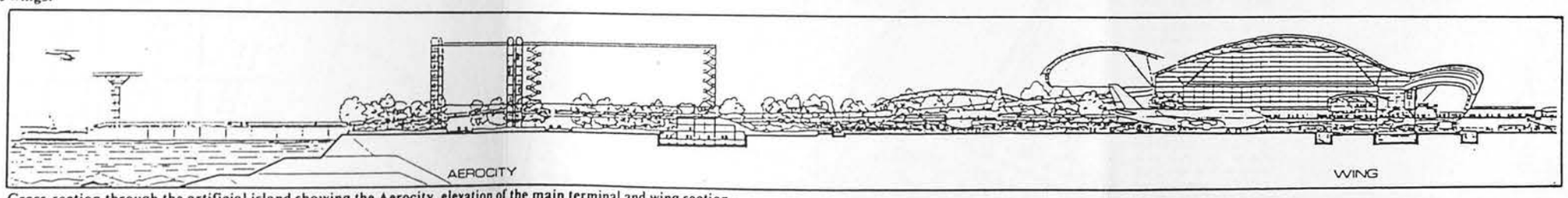


Sub-floor plan of the main terminal building and mezzanine plan of the extended wings.

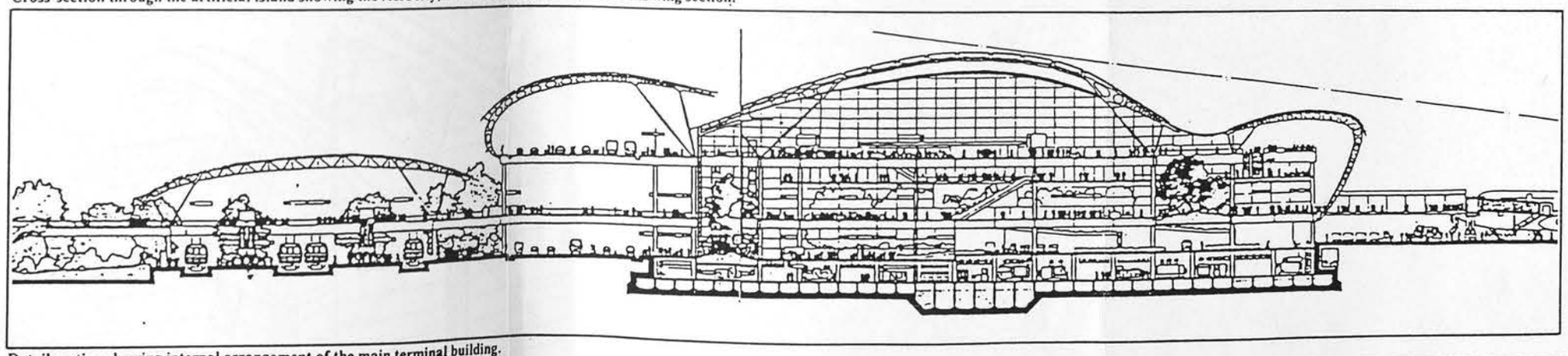


International departure floor level of the main terminal and roof-level plans of the wings.

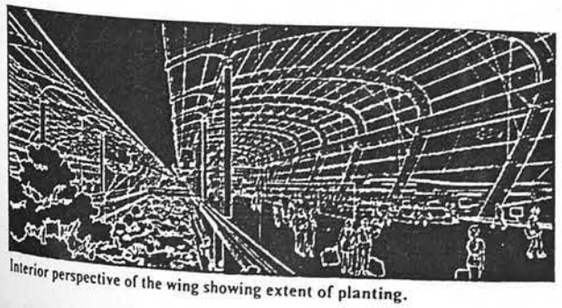
Kansai International Airport



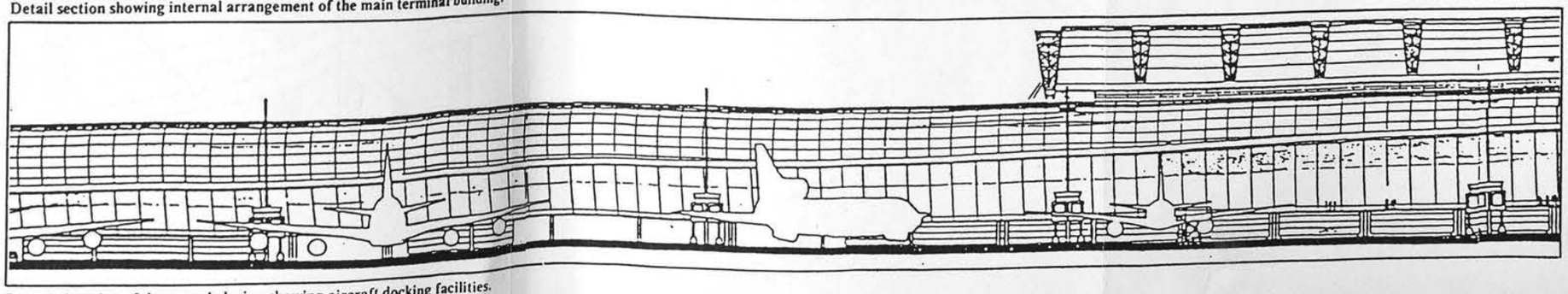
Cross-section through the artificial island showing the Aerocity, elevation of the main terminal and wing section.



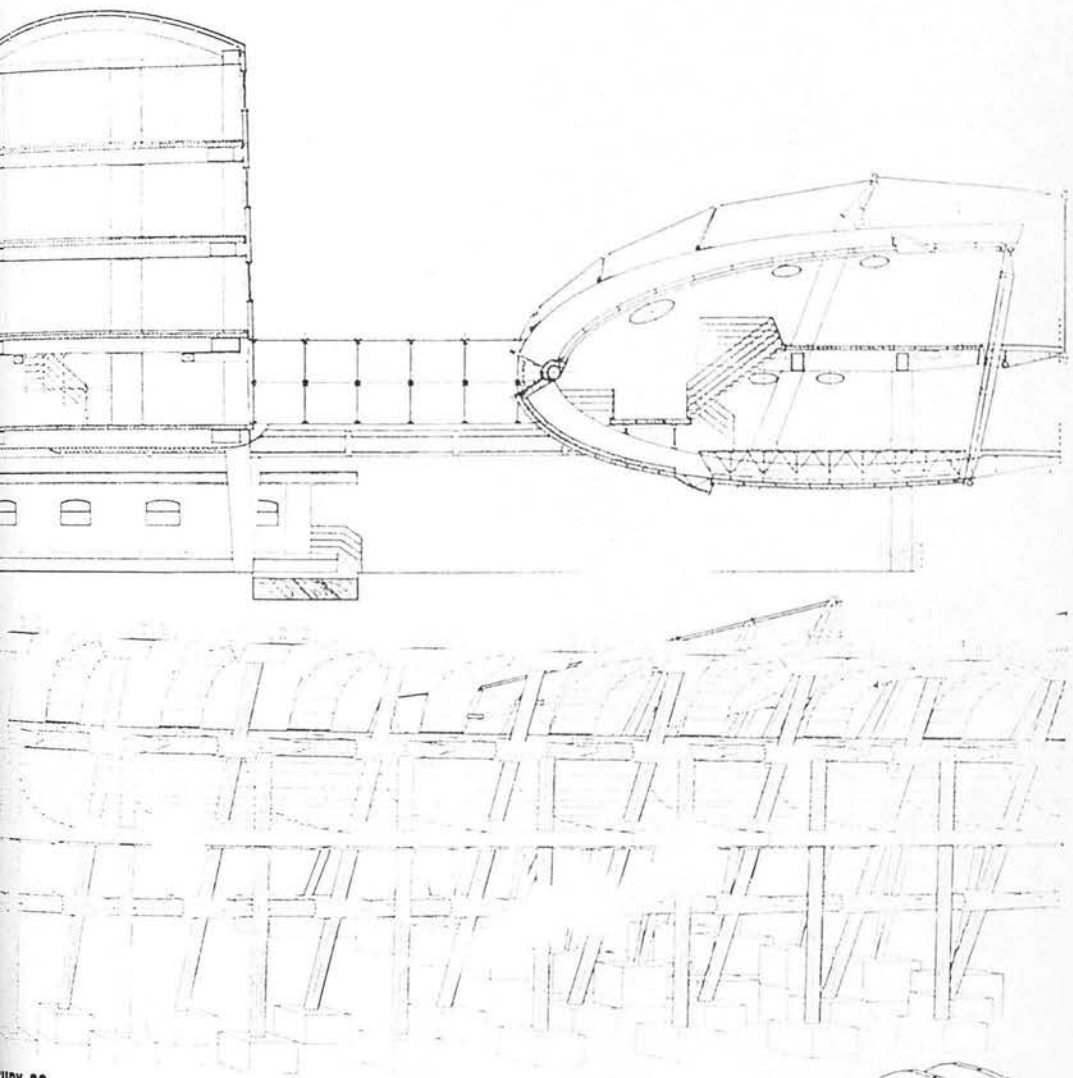
Detail section showing internal arrangement of the main terminal building.

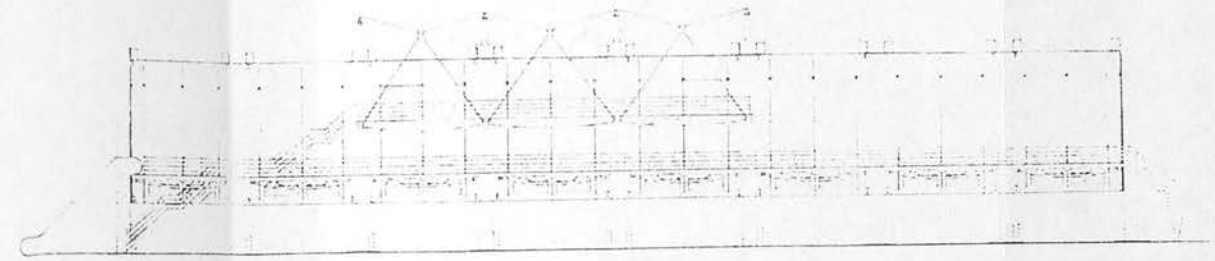


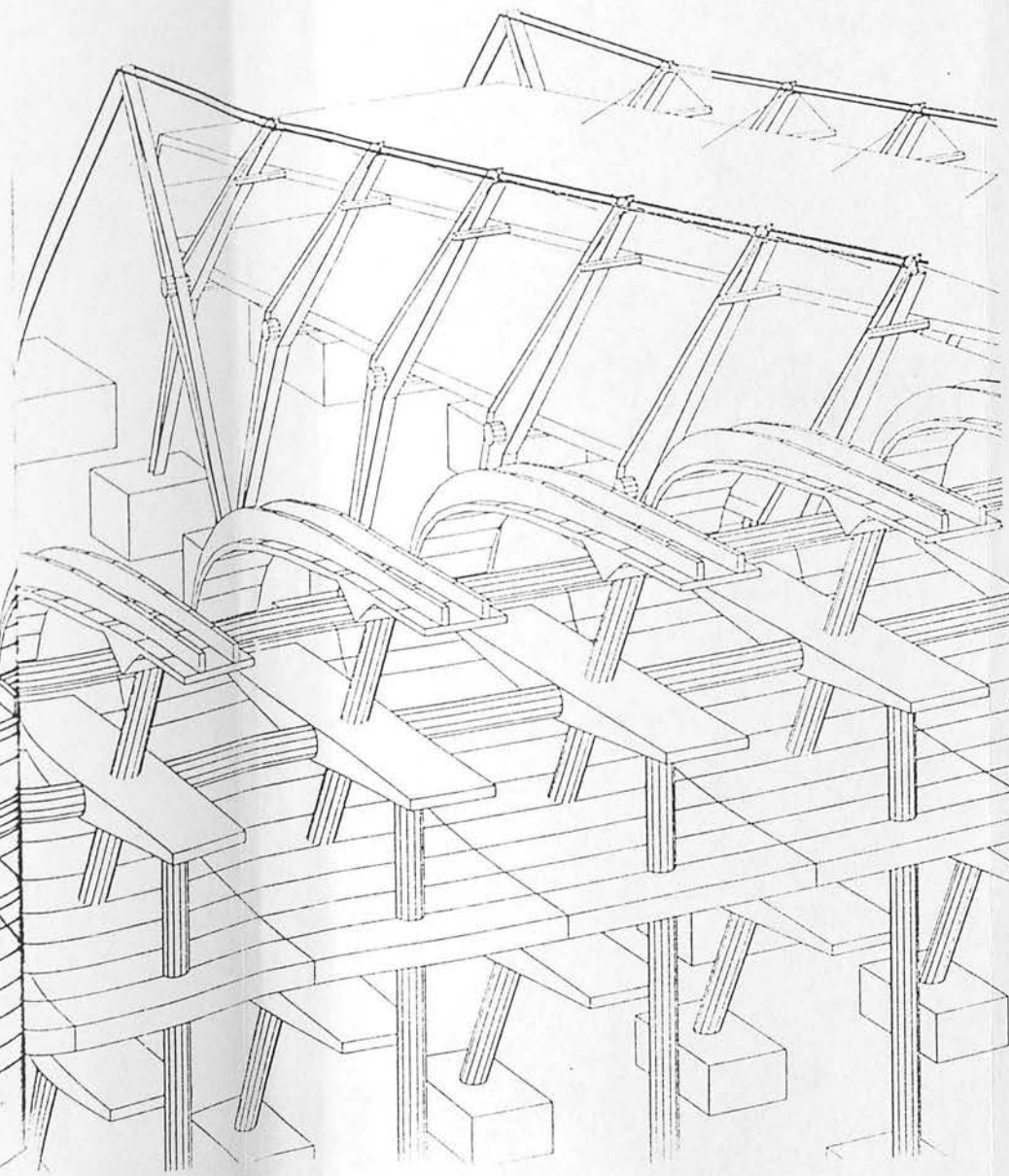
Interior perspective of the wing showing extent of planting.

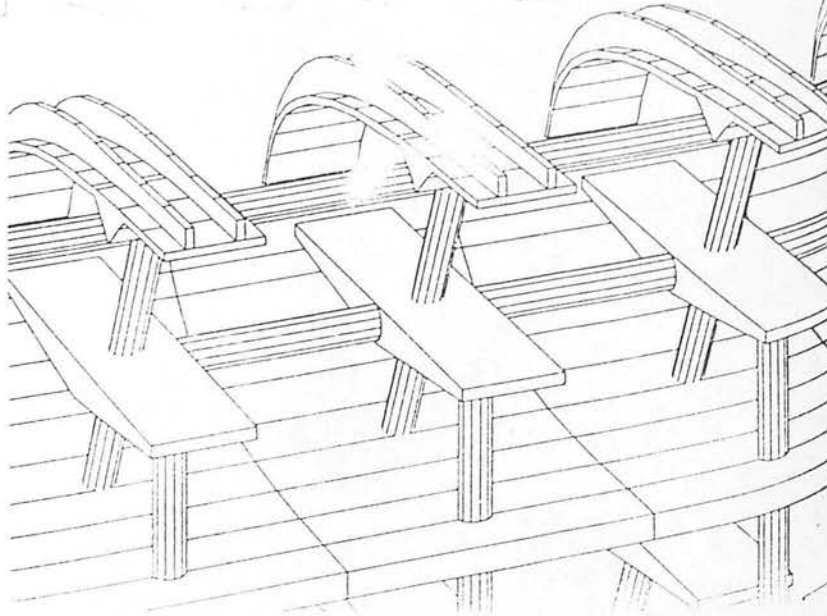


Partial elevation of the extended wing showing aircraft docking facilities.









UDY 32

MINAL
ES, WILL
OHN LYALL.
OVE ARUP
ERS

e World on the Seax.
toujours le nom de
du futur Terminal du
ellent hambourgeois
Londres, soit une
ropéen à une vitesse
ste. Outre qu'il va
s circulations à l'inté-
bourgoise, le Ferry Ter-
ssis le point de pas-
les voyageurs venus
chissant l'Elbe pour
ope de l'Ouest. On
ts ces conditions que
lauréat du concours
la ville, a eu toute lati-
er à ses plates-formes
plus spectaculaire pos-
rence en l'occurence
les bateaux. Hélas, des
dégâts
ont obligé
les à abandonner le
ny Terminal aura une
une coque en béton.

Heureusement l'aluminium poli et le zinc viendront recouvrir les ouvrages, comme des ecailles sur un - etrange - poisson.

● The project was the winner of a competition set by the Planning Department of Hambourg and financed by the ferry company DFDS. The site is on an existing spit in the River Elbe which is to be joined to the mainland by a land reclaimation project already nearly complete. The brief makes three main demands :

- A new ferry terminal building providing all the amenities expected of modern international terminals.
- Office accommodation for the operating Ferry company DFDS Seaways.
- A comprehensive plan for parking and queuing of passengers cars and freight containers.

The competition winning design accommodated office and ferry terminal space within the same building, but the revised scheme has separated the two areas into two separate structures, separated by a strip of water to emphasise the act of passing through customs control

onto Germany. The form of the building is obviously drawn form forms of ships and nautical tures. The first design was entirely steel structure. This been modified to a steel and crets structure and most rece for economic reasons to an ent pre-cast concrete structure. building is raised on legs to pre damage from flooding which oc occasionally on the site an to a car circulation beneath. The un side has a skin of polished alu ium, and the roof is a translus membrane. The front (south) el tion, looking onto the wate inclined curtain-wall glazing. office building is also conc framed but clad in natural-fi zinc sheet on walls and roof. F areas are beneath the office b ing and enclosed by glass walls two buildings are connected b glazed bridge (Section 3). The ject will be built in three phases first phases (as shown on the pl is to total 2 500 m²; the sec phase a further 2 500 m²; and third phase a further 5 000 m incorporate a terminal for a cruise service soon to be in duced. After all three phases built the structure will be 300 m with the office blocks in sepa units located behind as Phase 1.

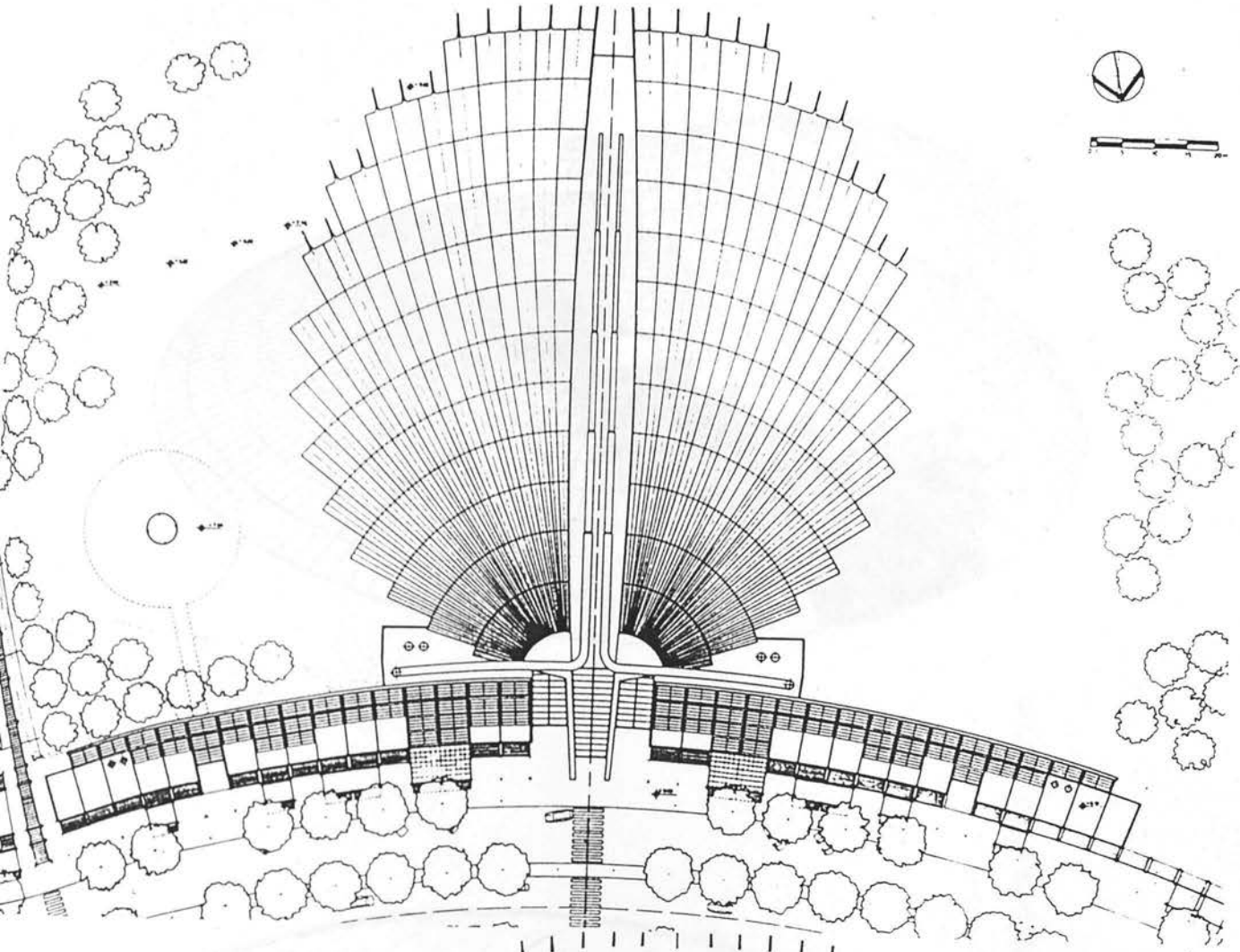
CASE STUDY 33: RAVENNA STADIUM

Architect: Renzo Piano

This new sports centre is located next to an existing football stadium, stands in a suburb of Ravenna and comprises two buildings. One is a gently curved building which follows an adjacent boulevard, and contains all the service related facilities needed by those using the centre. The outer building is a gymnasium with a capacity to seat 4,000 people, roofed over with two large shell forms.

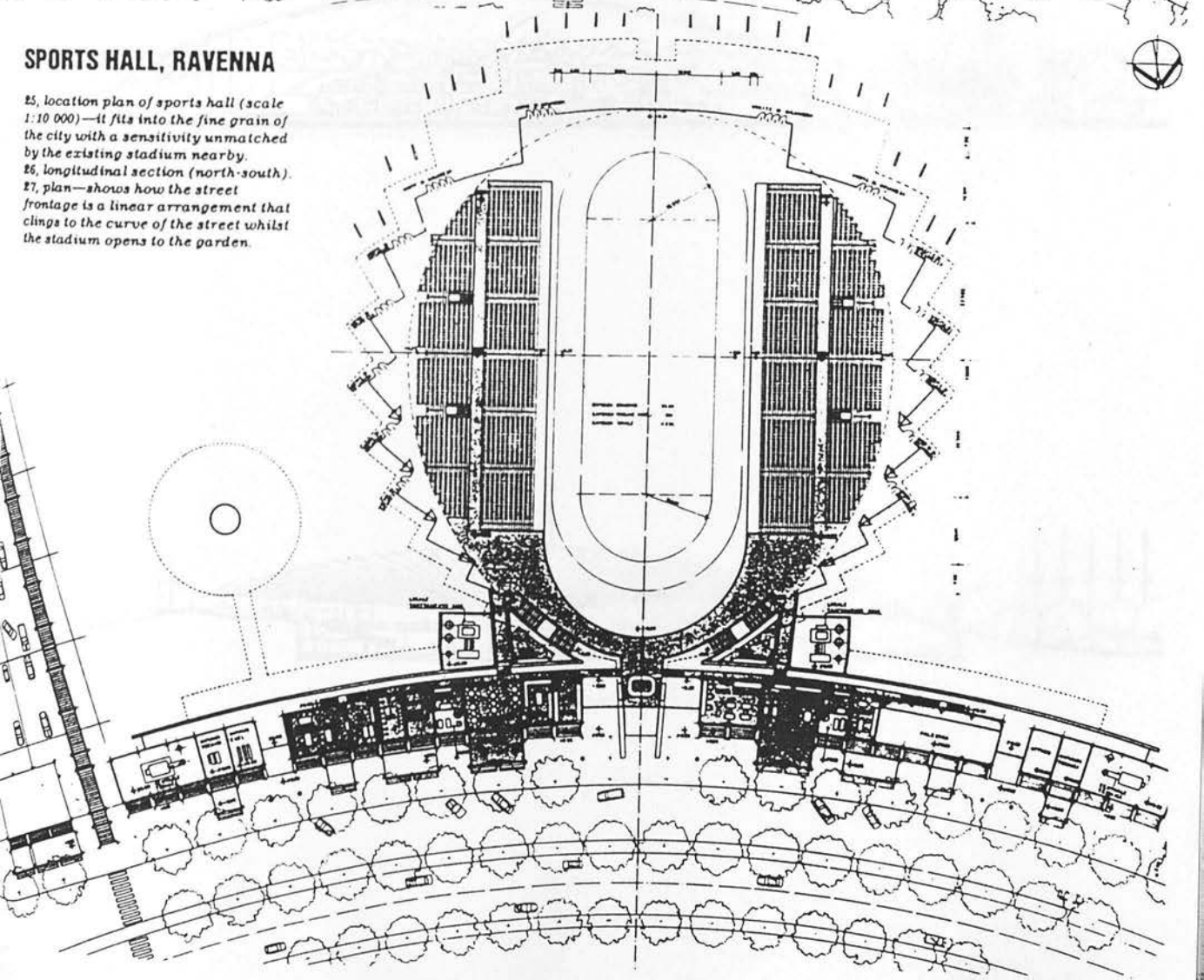
The main entrance is positioned where the two buildings meet and it is an important focus of the scheme, not only from a symbolic point of view, but also in terms of function and construction, too.

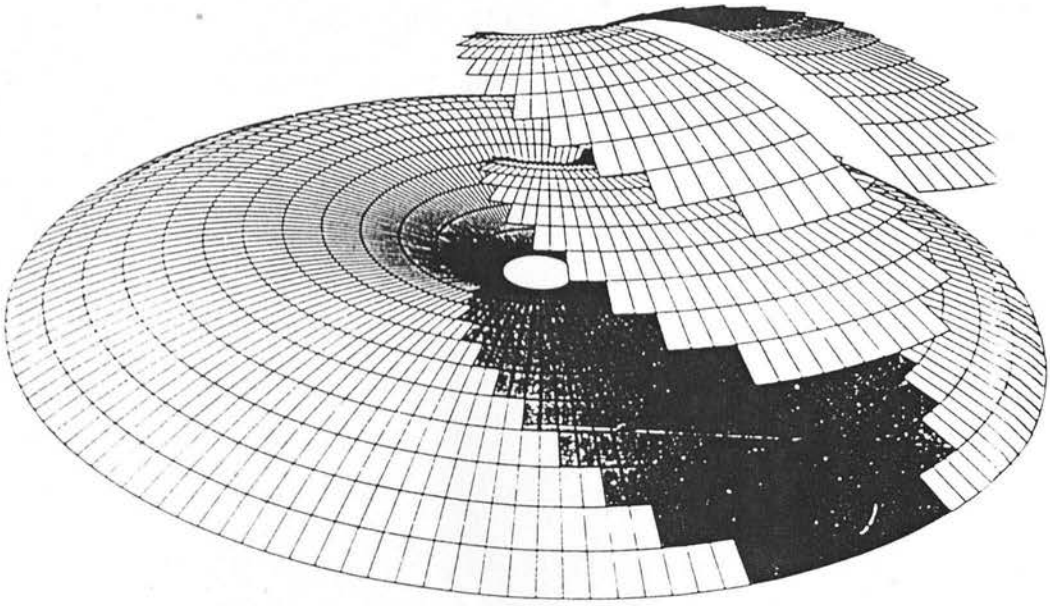
The form, is derived from the geomtry of the toroid. Between ten and twenty of the V-shaped, prefabricated beams were cast in a mould. Each beam is about eight metres long, 60 cm wide near the centre and gradually increasing in width toward the outside, to a full width of 2.5 m. This arrangement of beams was designed so that it was possible to balance the weight of each beam, by gradually changing the thickness of the members.

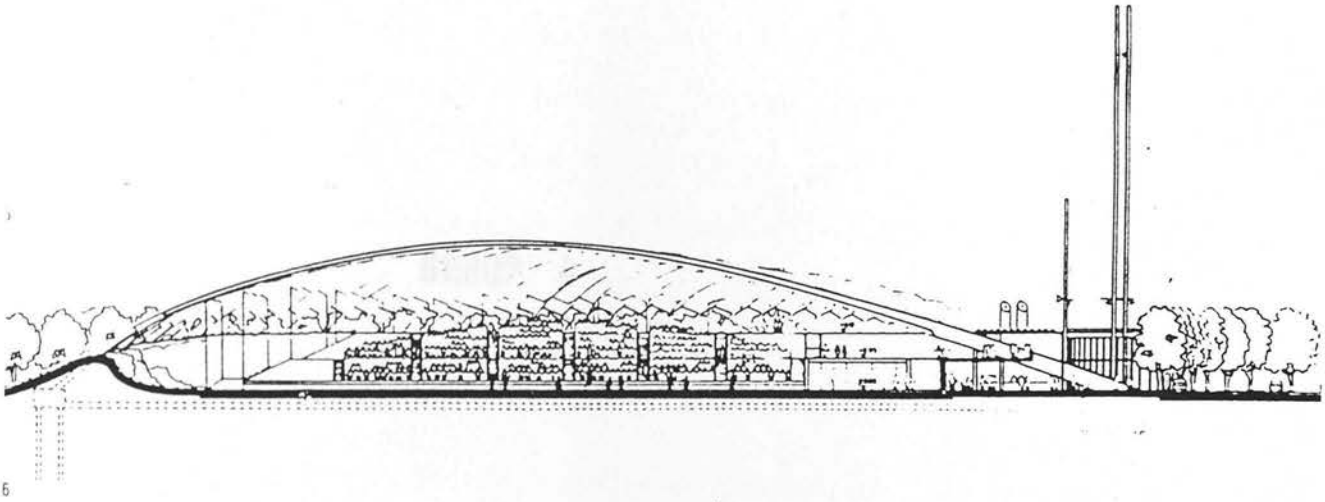


SPORTS HALL, RAVENNA

25, location plan of sports hall (scale 1:10 000)—it fits into the fine grain of the city with a sensitivity unmatched by the existing stadium nearby.
26, longitudinal section (north-south).
27, plan—shows how the street frontage is a linear arrangement that clings to the curve of the street whilst the stadium opens to the garden.







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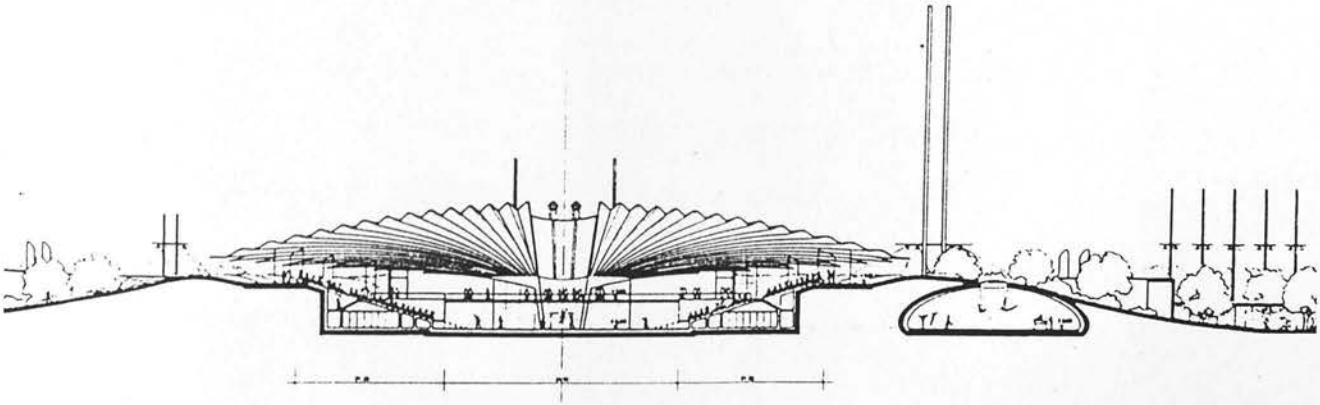


GROUP 4

CASE STUDY No. 34

Fujisawa Stadium

Extracts from critiques by Hiroshi Watanabe and Riichi Miyake

Architect: Fumihiko Maki

Engineer: Kimunt S.E.

1.0 Background

The city of Fujisawa is located some thirty kilometers from Tokyo. It is an important satellite of the capital, to which many of its residents commute. In recent years, new industries have moved to the area and the city's population has increased to two hundred and twenty thousand. In 1980, the city announced its plan for a park, centered around public sports facilities, to be located 6.5 kilometers from downtown Fujisawa. The first stage of the project includes this gymnasium, a soccer field, and outdoor pool, and an archery field.

This gymnasium is expected to be the nucleus of the central municipal sports park. It consists of two wings; a main arena with a seating capacity of two thousand and with space for three volleyball courts and a subarena wing. The main arena, like many Japanese gymnasiums, has a stage making it possible to use the space for purposes other than for sports. In the basement of the four-story subarena building is a sauna. On the first floor are administrative offices and training room; on the second are training rooms for judo and kendo fencing plus restaurants. And on the top floor is the arena itself.

2.0 Design Philosphy

Fumihiko Maki's architecture is characterized by a distinct sense of lightness. In the intersection and succession of external and internal spaces he emphasizes the expansion of dimensions, demarcating the space by successive transparent surfaces so as to create a mood of infinite clarity. Such is the case in this work, although the architect's concern is primarily to separate and diffuse each individual part of the buildings in such transparent spaces. Each element seems to float without mutual relation, detached from the skin and surfaces which define the architectural integrity of the buildings.

Indeed, this complex displays a peculiar disposition of forms, in which every part asserts itself separately from the total whole: the projecting seats for the spectators in the main arena that remind us of the forms of medieval armour; the northern stage that is the very focal point of the internal space; the distinctive form of the trusses supporting the superior construction; the gigantic crescent-shaped concrete podia serving as the base of these steel members, and the independent air-conditioning shafts are expressed individually.

3.0 Expression of Form

In the design of both stadiums, ie the Tokyo Metropolitan and the Fujisawa, Maki intentionally fragments the elements which make up the form of the building. Thus, in the Fujisawa, the axes of small arena is at an angle to the main arena, the hovering forms of the arena structure and their serrated profile contrasts with the idea of rectilinear, axial composition. The columns which

support the grandstands are concealed by tile-clad walls on the east and west ends, thereby creating an impression of supporting buttresses. There is a marked contrast with this building and Tange's Olympics Stadium built in 1964. The flowing, continuous forms of Tange's stadium were intended to express national unity whereas Maki's fragmented forms were intended to express the pluralism in society today.

Maki has recently written about the peculiarity of traditional Japanese space: successive layering of space, the appreciation of which enabled him to achieve a much more dynamic manipulation of architectural volumes. Strolling through this complex, one is fascinated by the successive scenes encountered, elements and details appearing one by one, but disposed in an unusual and perplexing way. Upon entering the sub-arena, guided by two rows of columns of different sizes in the transparent entrance hall, one suddenly discovers a the compressed space with a staircase of medieval character. This abrupt expansion and compression of space enhances the layering of internal spaces, a technique that is characteristic of the architect.

The manipulation of space in this manner leads to a concept of crevices appearing between the volumes. One example appears along the roof framework of the main arena in the form of pleats; another as a slit dividing the stainless forehead of the sub-arena in half. The many crevices of irregular shapes around the northern stage and between the pointed roof and the interior wall of the basketball court are nothing but folds that 'sew up' spaces and fasten together different elements. To underline

this effect, Maki accentuated the presence of the crevices, which at night cuts sharply through the darkness with a streak of linear light.

4.0 Structure

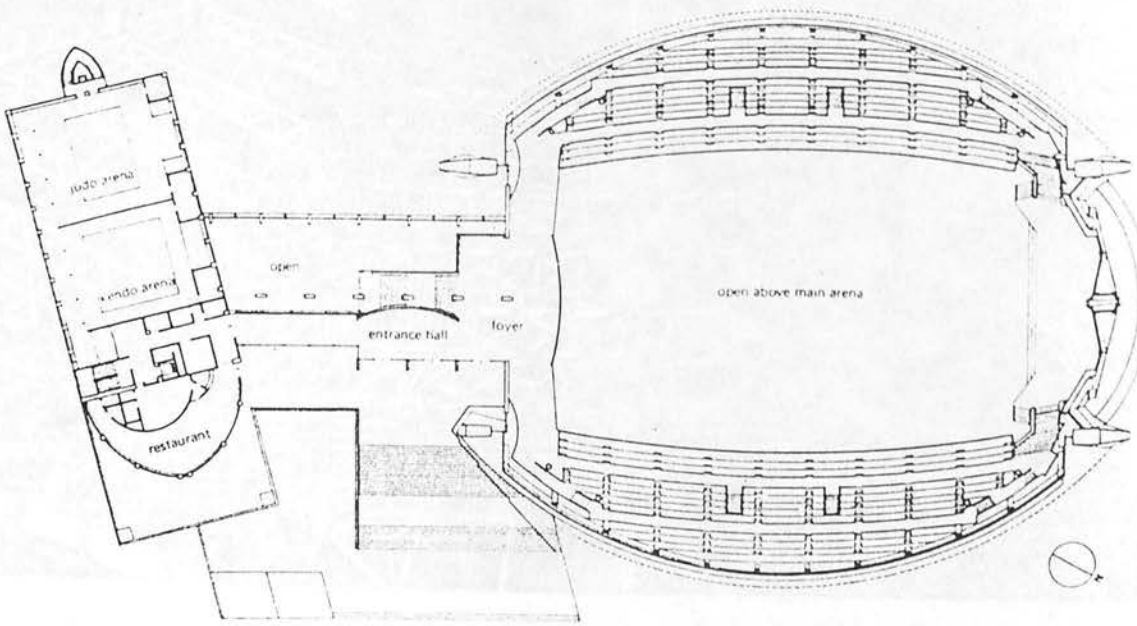
The central facility is the main arena which seats two thousand spectators. A stage to the north provides a focal point for the huge space. Two giant arches span this space. These arches have a triangular section with a base measuring 3.5 metres, a height of 3.5 metres, and are composed of H-section steel members. The arches are 80 metres long and have a 2-meter rise at the top. Prestressed beams tie the arches underground. Large crescent-shaped concrete podia supporting one thousand spectator seats are cantilevered on both side of the court. There are girders at 6 meter intervals between these podia and the two arches. A compression ring along the top edge of podia absorbs the horizontal thrust of these girders. Thus all members are under compression.

The second wing has a smaller gymnasium, called a sub-arena, on its top floor. It is 12 metres wide, with a roof structure based on a pointed arch. Below that level, on the second floor, is a restaurant and a practice hall for Judo and Kendo. The restaurant is located at the rounded end of the building and glass screens at its top provide a visual connection to the sub-arena. The first floor contains a training court, a conference room, office, and other administrative spaces.

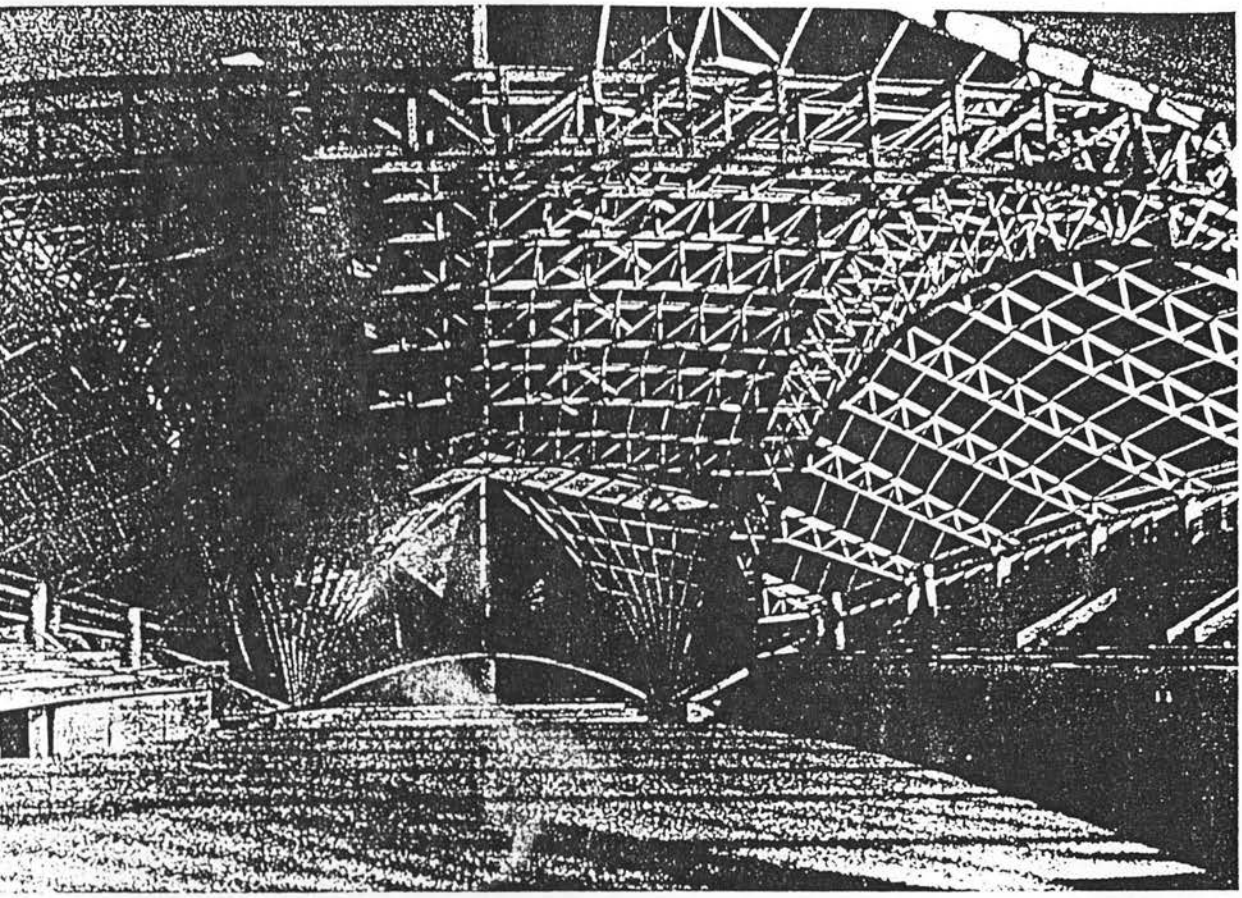
A partly two-storied space with a rectangular plan connects the two wings. The entrance and lobby for those using the gymnasium is located on the first floor. A broad open stairway leads to the second floor lobby, which is used by spectators for access.

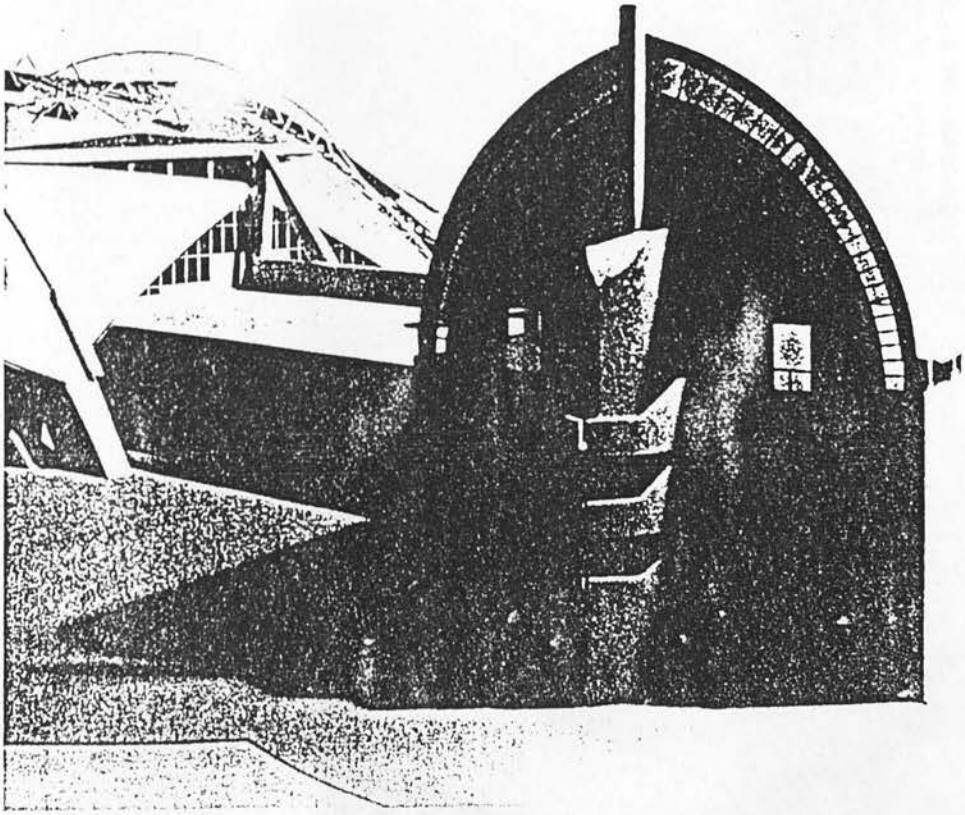
The roof, which has a vast arch with a span of 80 meters between support points and is intersected by a lattice structure of H-form steel, is covered with stainless-steel sheeting 0.4 millimeters thick.

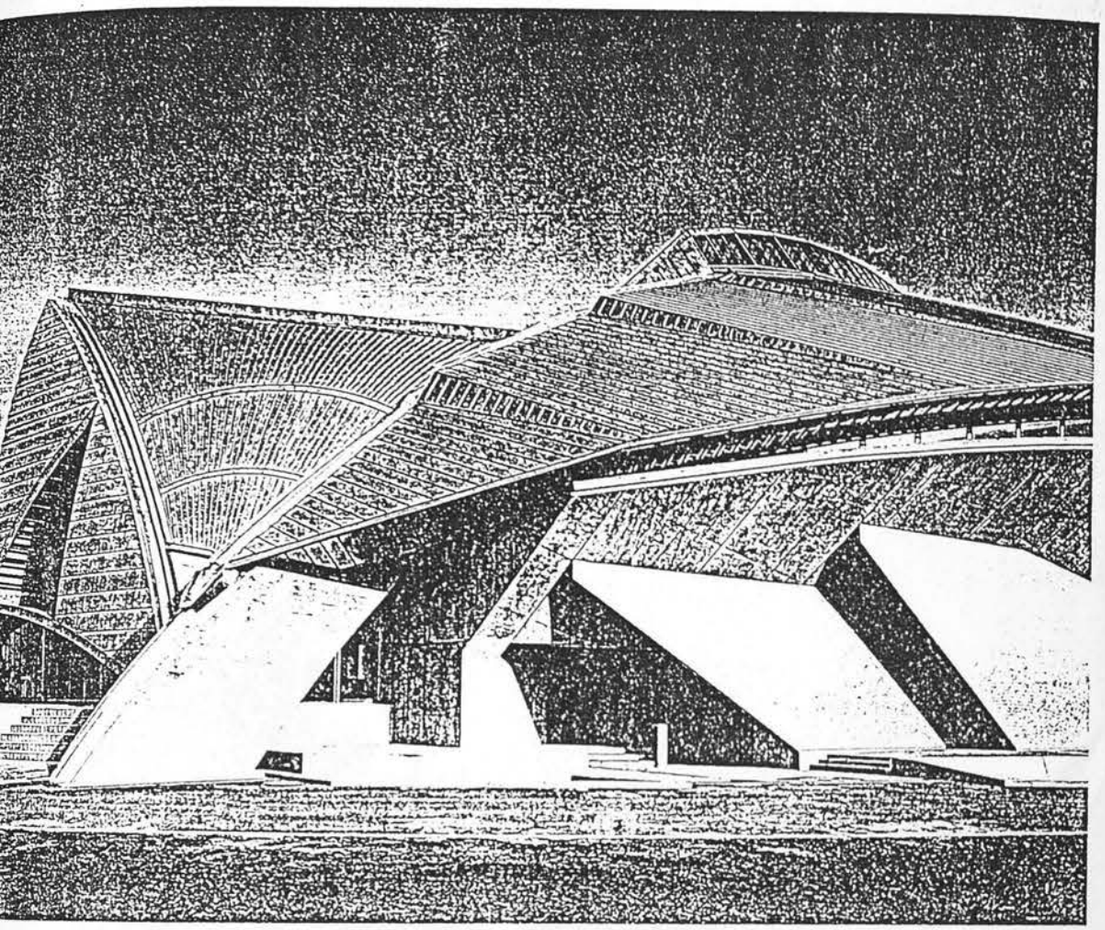
Data
FUJISAWA MUNICIPAL GYMNAGIUM/Fujisawa, Kanagawa Prefecture; architects: Maki and Associates; structural engineers: Kimura Structural Engineers; mechanical engineers: P.T. Morimura & Associates; site area: 64,105m^2, building area: 6,522m^2, total floor area: 11,647m^2; structure: reinforced concrete and steel frame, 1 basement, 3 stories, and 1-story penthouse; completion date: September, 1984.
Photos: Taisuke Ogawa, Photography Dept., JA



Second-floor plan; scale: 1/1,000







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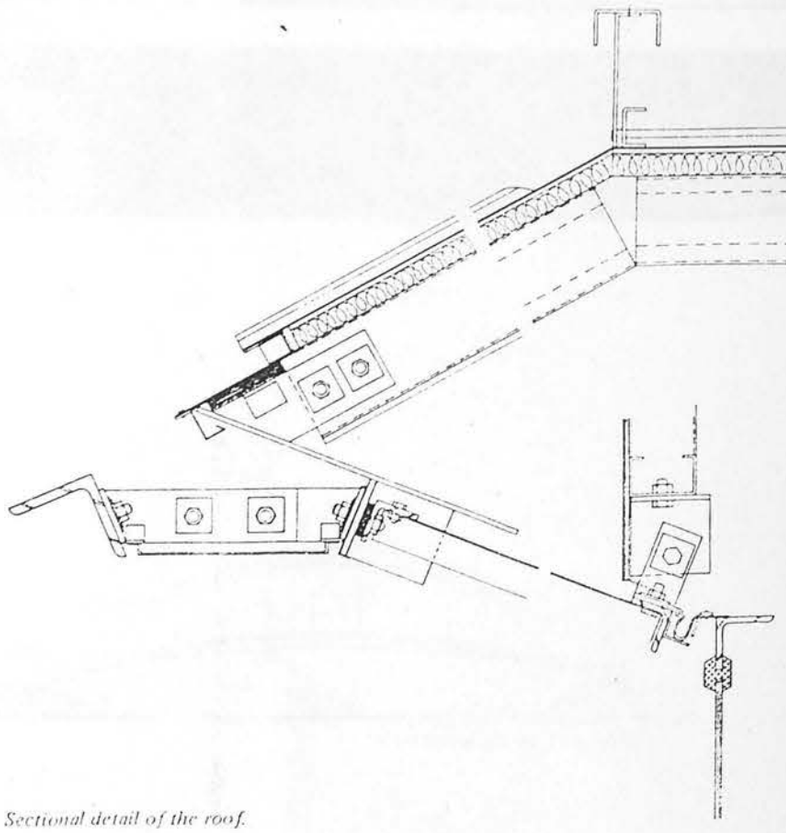
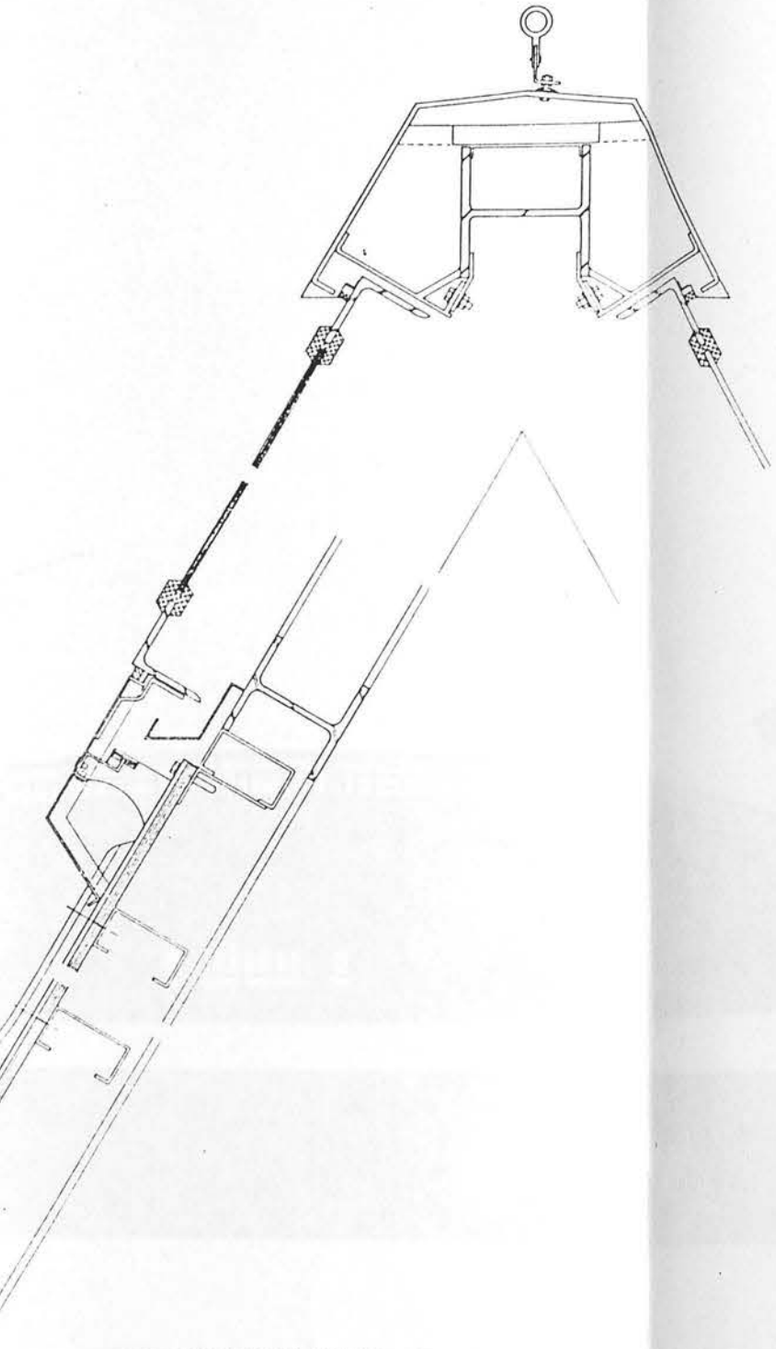
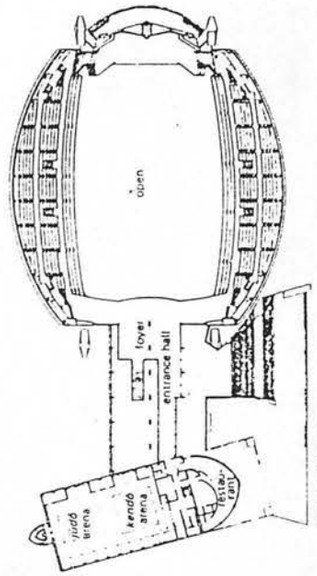


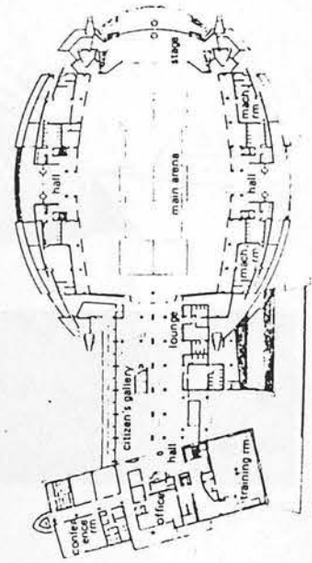
Sectional detail of the roof.



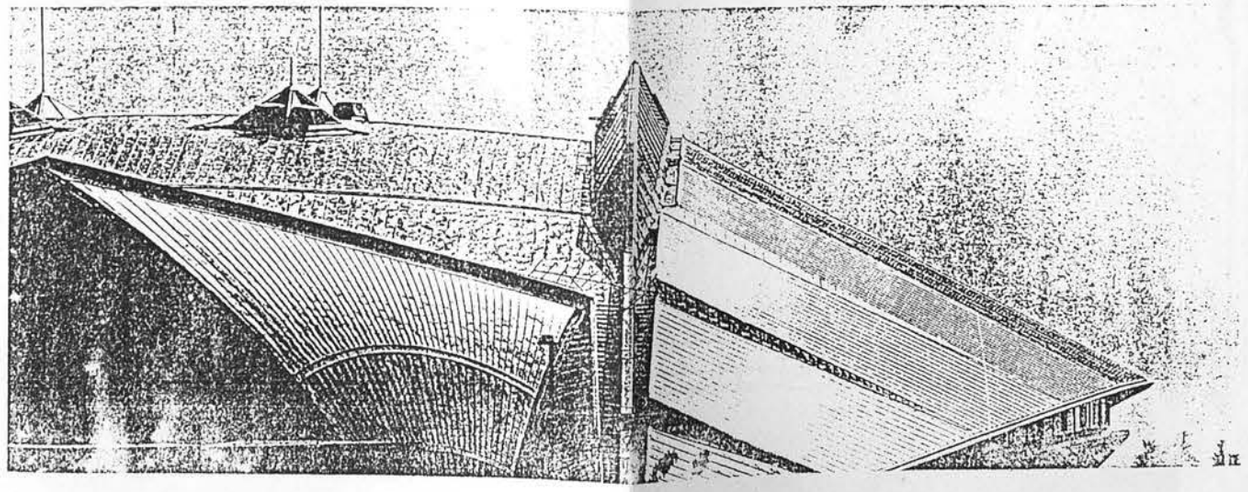
Third floor.



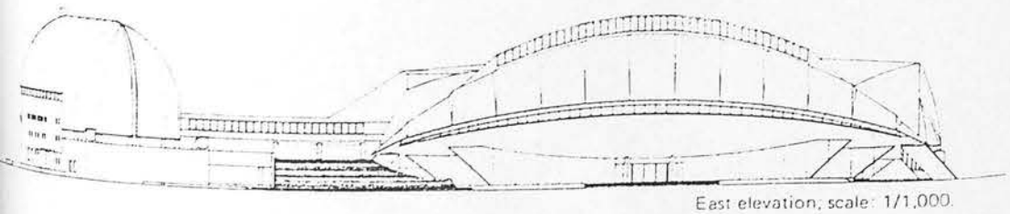
Second floor.



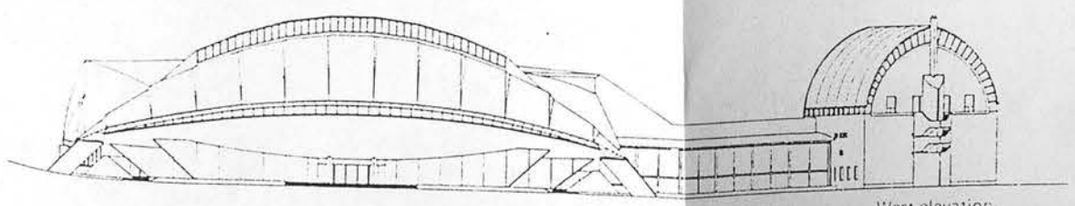
First floor, scale: 1/1,800.



General view from the east.



East elevation, scale: 1/1,000.



West elevation

CASE STUDY 35

(Extract from an Article "Metropolitan Sports Centre, Tokyo for 1990" by Philippe Vernier, I'arca, November 1987)

1.0 Background

This is a reconstruction of Tokyo's Metropolitan Sports Centre built in 1956. The site area of 45,800 square metre is located near to the major parks of central Tokyo, and is adjacent to a railroad station. A circulation route for large groups of spectators is provided between the existing national stadium and the site.

The total floor area of the complex is 44,000 square meters and comprises three major buildings: a main arena, an indoor swimming pool, and a sub arena. The main arena and the indoor swimming pool will be roofed in stainless steel.

It is expected to be completed in April, 1990.

2.0 Form and Expression

The overall composition attempts to create a new urban landscape by juxtaposing strong geometrical, and symbolic forms.

The sports centre comprises a broad, articulated base four and a half hectares in area, on which the sports building, the indoor swimming pool, and a smaller sports building are expressed as three separate elements of the building form.

The larger building and the indoor swimming pool are roofed with steel sheets in order to reflect the voluminous character of the interior spaces. The other buildings are expressed in concrete, to express by means of texture, their relation to the ground. By contrast, the main building is a metaphorical cloud hovering above the ground.

The smaller sports building is shaped like a tiered quadrangular pyramid, crowned by a slender pyramidal lantern inscribed within a metal framework in the form of a truncated pyramid placed upside-down. The larger sports building, the most conspicuous, has a circular ground plan with its poles squashed on the main axis of the playing field, flanked by two seating stands in the shape of a double curvature segment.

The roofing of the main building acquires a Japanese character with a visual outline likened to traditional samurai helmets. Two curved pitches, both with a double inclination decreasing in the lower zone and with circular edges, move towards the top, forming a sort of pinnacle from which with an undulating movement, the lower roof surface descends longitudinally, filling the two areas left uncovered by the circular roof pitches.

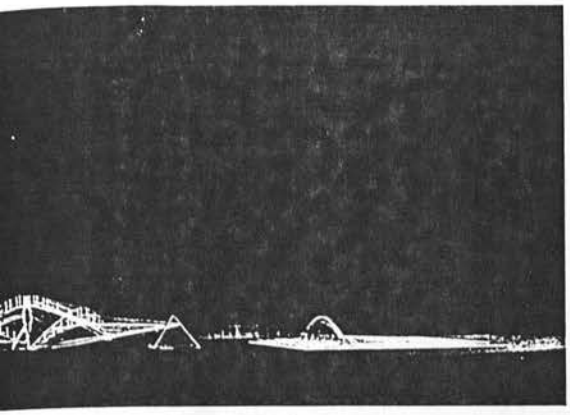
The division of the roof into five sectors, four of which are symmetrical to the central spine, has made it possible to fit low horizontal continuous slots between each sector, allowing for natural lighting and ventilation.

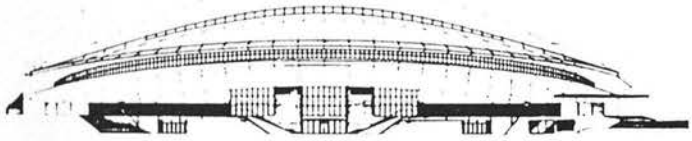
A three-dimensional metal frame, with a complex and multiple curvature design, supports the main roof surfaces. These are clad on the outside by steel sheets and on the inside by rectangular panels set transversally on the boundary lines between the different pitch slopes, so as to present a single surface ribbed with numerous different vanishing points within the interior.

The indoor swimming pool is a parallelepiped with a roof divided into flat and curvilinear zones, and with a large rectangular skylight and a transverse semi-circular arch section. Inside the building is a main swimming pool with three sets of seating stands on one side. A second, smaller pool for children is located at a level lower to the main pool.

Situated all around the three emergent volumes of the sports constellation, in the connecting base, are the numerous service and technical equipment rooms. This low construction is developed along a broken line enclosing a number of small uncovered protected areas reserved for the public.

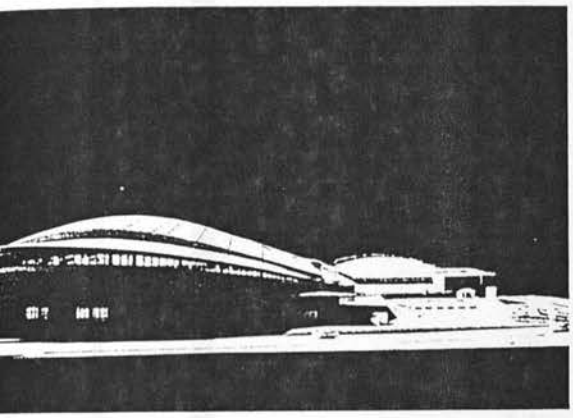
The overall design of the Metropolitan Sports Centre tries to reconstruct a new urban landscape with strong architectural signals connected to each other by a formally "neutral tissue" in which the public provides the most significant presence. The architecture is used here as a community attraction. It serves a functional and visual purpose, but also strikes and stirs the imagination by its complex organisation of space and form.

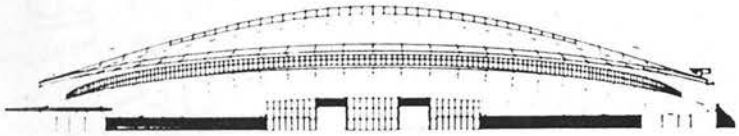




■ *Disegno prospettico al computer del complesso sportivo.*
■ *Perspective computer diagram of the sports complex.*

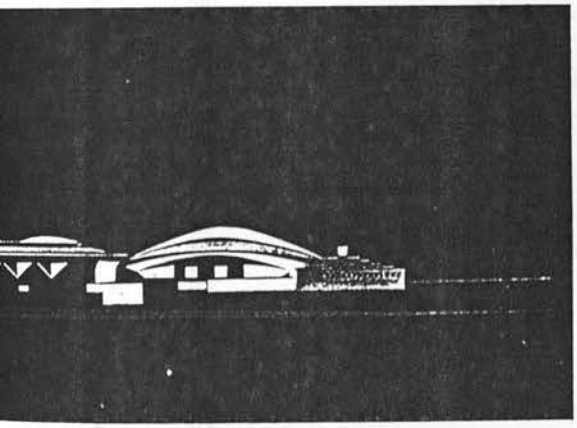
■ *Prospetto nord del Palazzo dello Sport.*
■ *North prospect of the Sports Palace.*

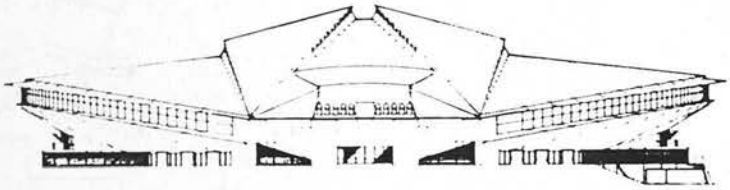




■ *Modello del progetto del Metropolitan Sports Center, visto da nord.*
■ *Model of the Metropolitan Sports Center project, seen from the north.*

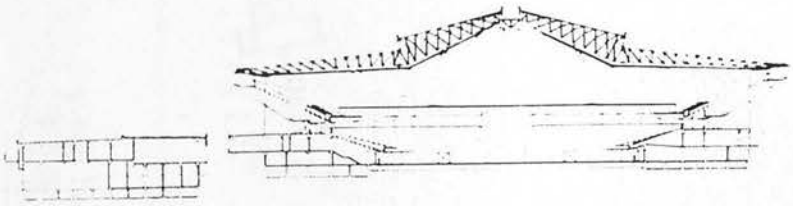
■ *Prospetto sud.*
■ *South prospect.*





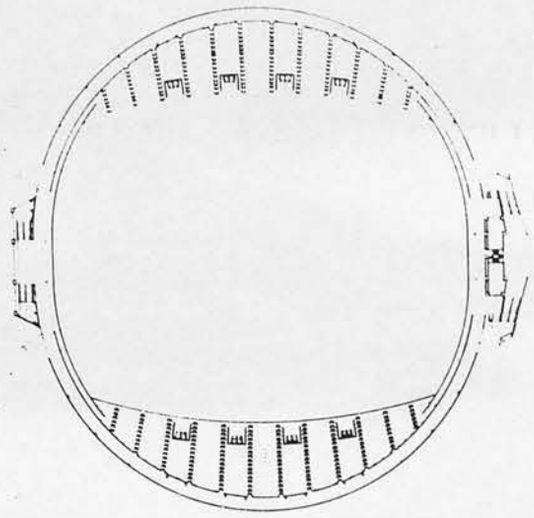
■ *Vista dello schema volumetrico del complesso, visto da sud.*
■ *View of the volumetric plan of the complex, seen from the south.*

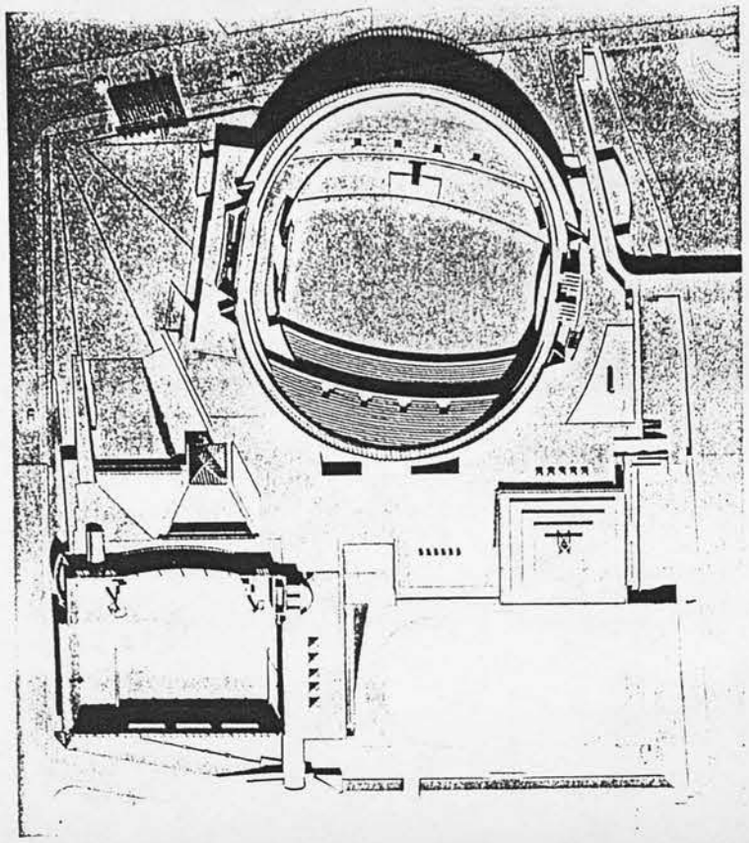
■ *Prospetto est.*
■ *East prospect.*



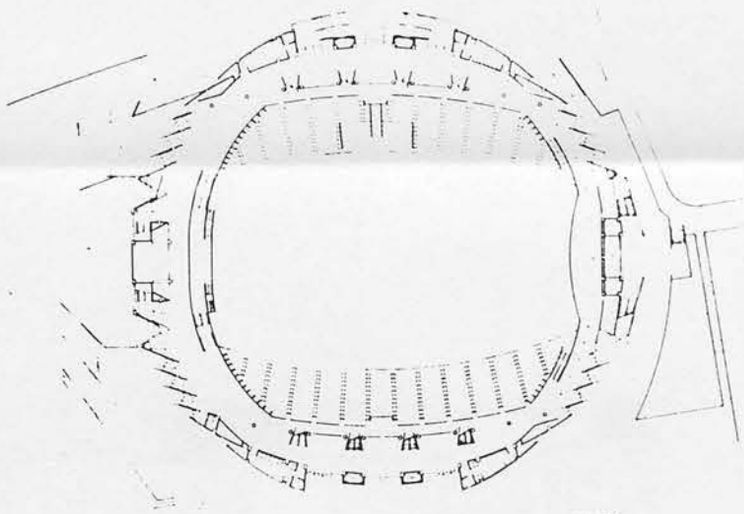
■ *Sezione nord-sud.*
■ *North-south section.*

■ *Pianta del Palazzo dello Sport al terzo piano.*
■ *Plan of the third floor of the Sports Palace.*



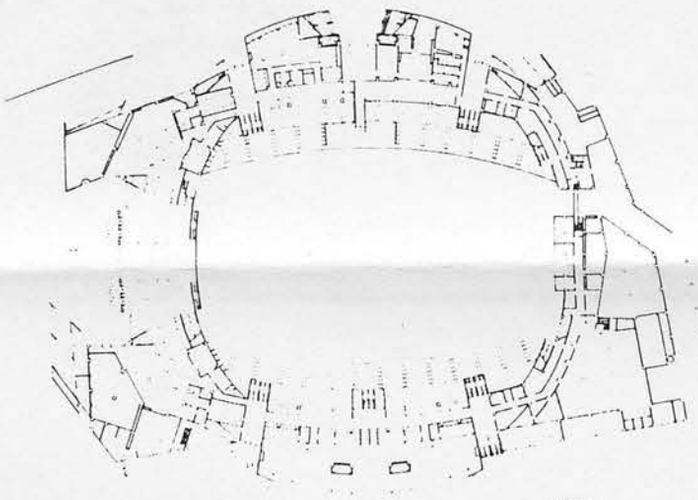


■ *Pianta del secondo piano.*
■ *Plan of the second floor.*

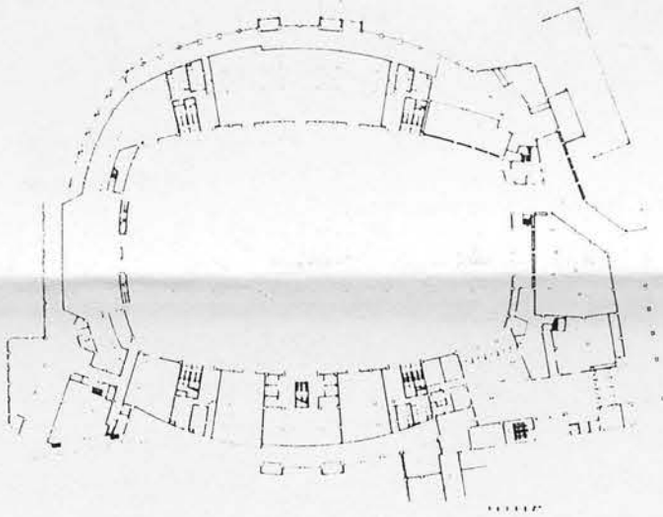


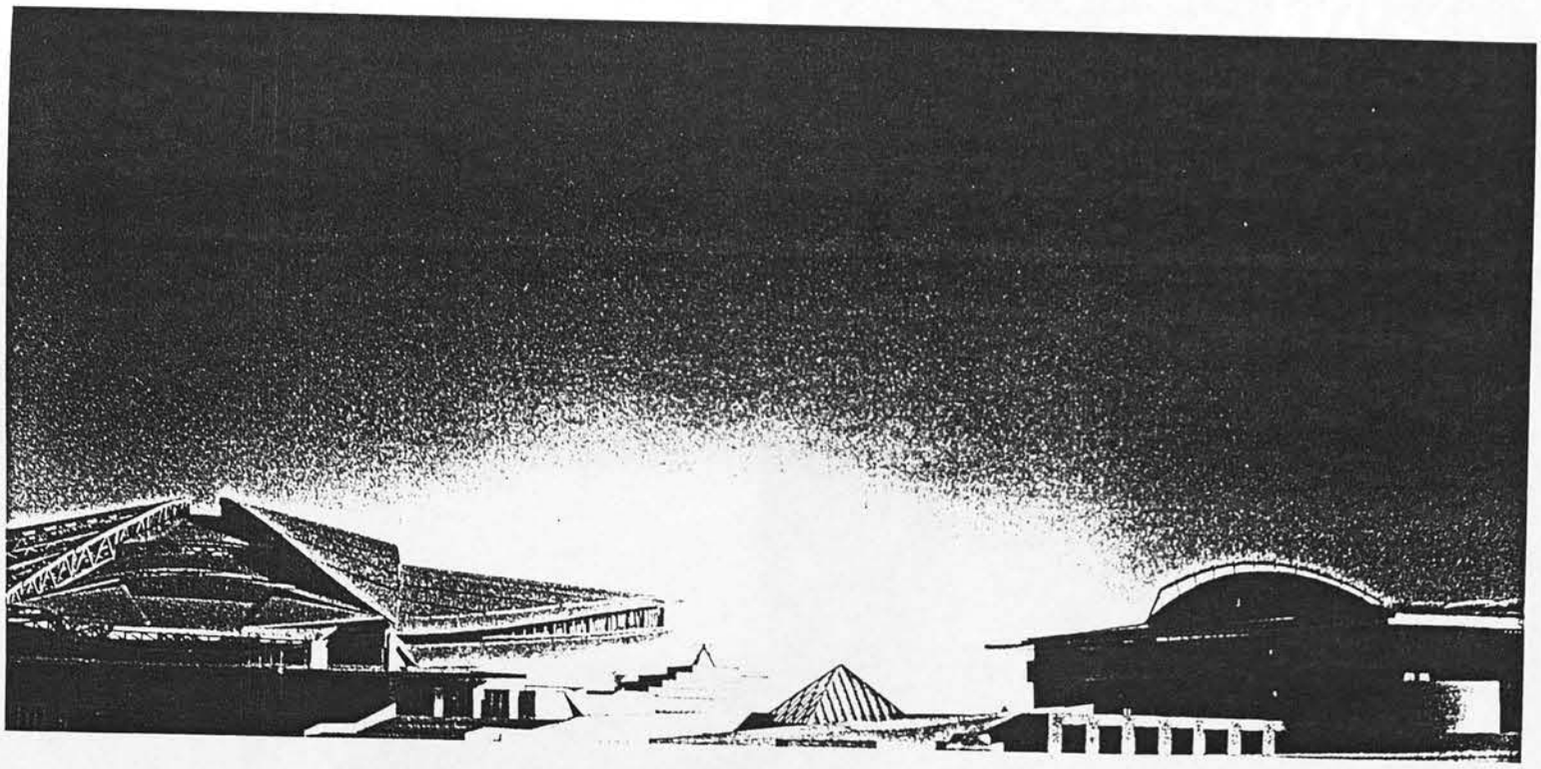
■ *Vista zenitale del modello del Metropolitan Sports Center.*
■ *Zenithal view of the model of the Metropolitan Sports Center.*

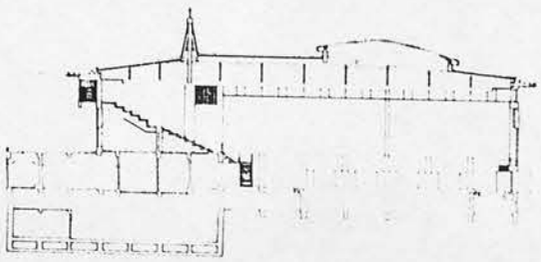
■ *Pianta del primo piano.*
■ *Plan of the first floor.*



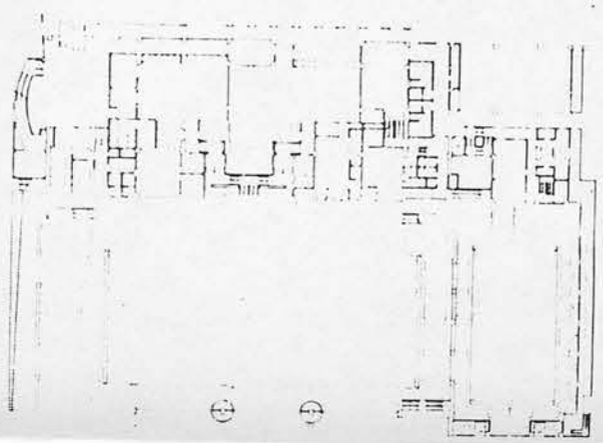
■ *Pianta del piano terra.*
■ *Plan of the ground floor.*



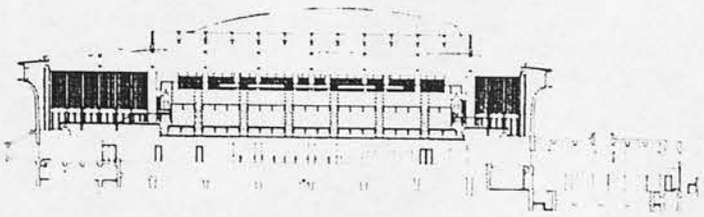




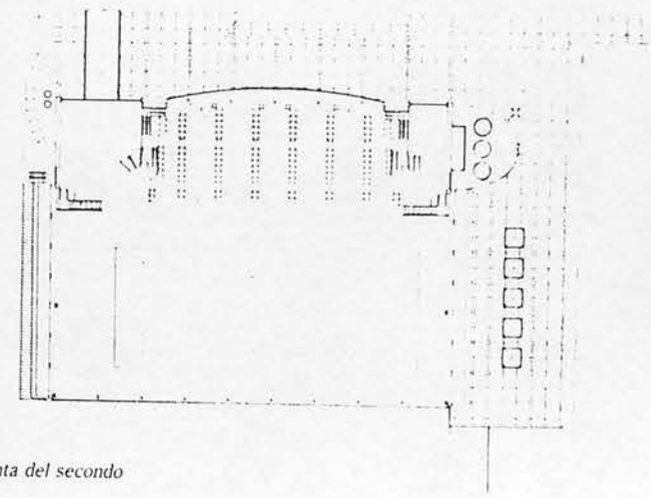
■ *Sezione trasversale del Palazzetto dello Sport con la piscina di 50 m.*
■ *Cross-section of the smaller Sports Palace with the 50 m pool.*



■ *Pianta del primo piano.*
■ *First floor plan.*



■ *Sezione longitudinale con la piscina di 50 m nel corpo edilizio principale e un'altra di 25 m nell'appendice inferiore.*
■ *Longitudinal section of the 50 m pool in the main building and the 25 m pool in the lower extension.*



■ *Pianta del secondo piano.*
■ *Second floor plan.*

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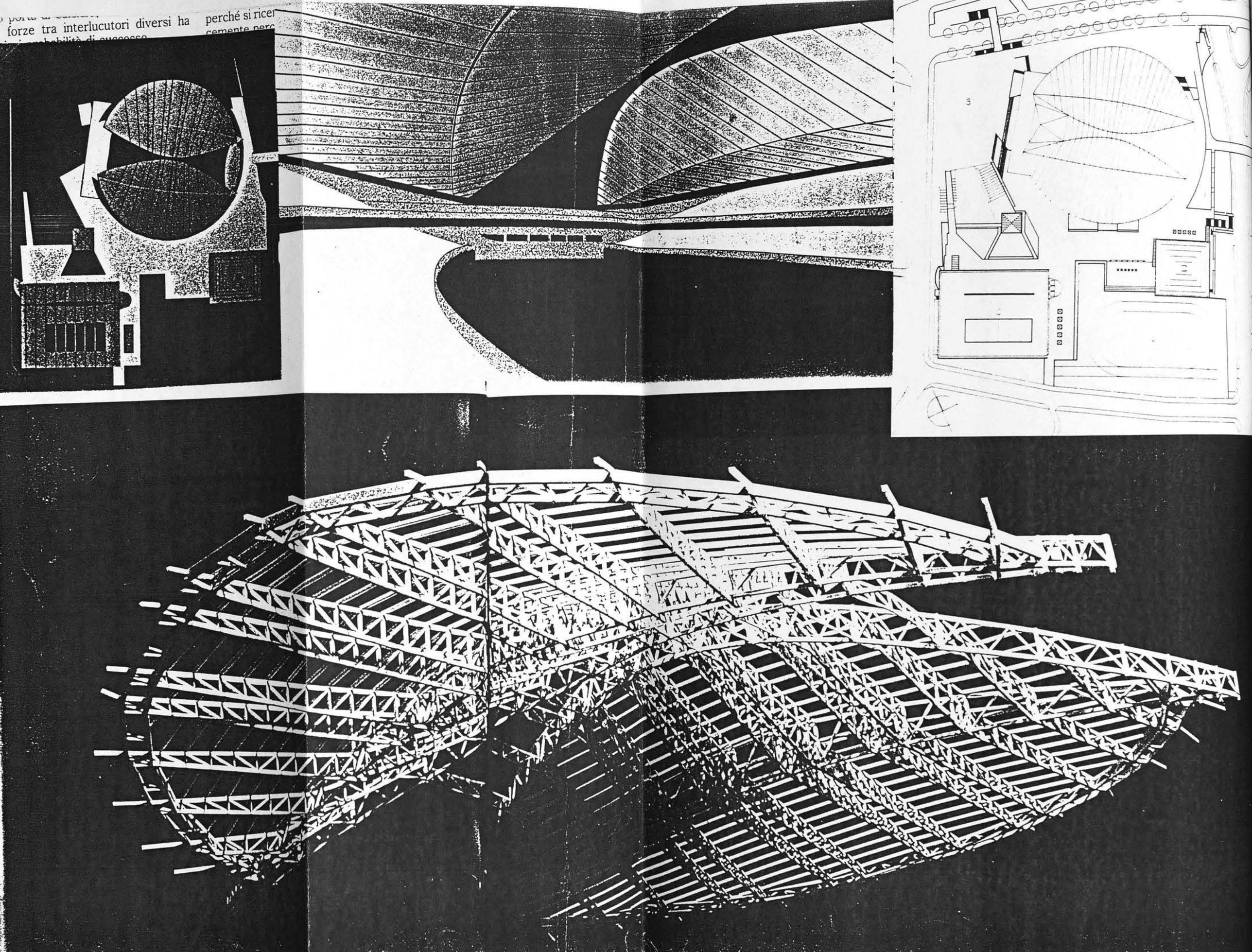
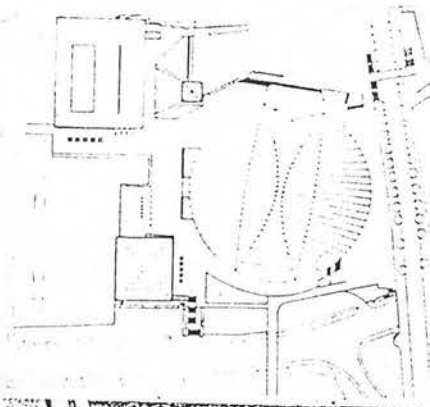


Tokyo Metropolitan Gymnasium

Shibuya Ward, Tokyo



CASE STUDY No. 36

BARI STADIUM (ROOF STRUCTURE)

Architect: Renzo Piano

Engineers: Ove Arup & Partners

Extract from a report by Tristam Carfrae, OAP 18/8/88

1.2 Description of Structure

The roof is a continuous cantilever roof surrounding the whole pitch and covering the seats in the upper tribune. It arches up from the back of the tribune structure towards the centre of the stadium, starting off at forty five degrees and ending up level at its inner edge. The span of the cantilevers reduces from twenty six metres either side of the stadium to fourteen metres at the ends. The height of the roof also reduces from the sides to the ends.

The roof supporting structure is made of steel with some elements in stainless steel. The roof cladding consists of a pre-tensioned, doubly curved, fabric membrane.

The roof is symmetrical about the two principal axes of the stadium and is therefore made up of four similar quadrants. Each quadrant is divided into six typical bays and an untypical bay which supports the scoreboard at the end of the stadium. This untypical bay is located midway between two quadrants.

Each bay is made up of a main bay section and an infill section

between main bays - refer to figure 1.

In the main bays the membrane is divided into four panels. In between the panels are three rod braced, tubular arches which span from bipods mounted on the concrete tribune structure at the rear,to the rear edge of an open 'U' shaped tubular truss at the front. The truss in turn spans between the tips of two box girder cantilevers which are bolted directly to the concrete tribune structure. These cantilevers form the edges of the main bays and support the edges of the two side membrance panels. The rear edge of the membrane is attached to a horizontal beam spanning between the bipods and connecting to the cantilevers at either end. The front edge is connected directly to the lower rear edge of the 'U' shaped truss. Running parallel to the truss and rear beam are four intermediate roof struts which run from cantilever to cantilever and stabilise the braced arches.

The infill bays have a membrane which is divided into five small panels. In line with the main bay intermediate roof struts are four small arches which span from the box cantilever on the edge of one main bay to the cantilever on the edge of the next bay. These arches are stabilised by rods in a diamond pattern, which run from the midpoint of the arches to halfway between their springing points on the cantilevers. Three membrane panels span between arches with their other edges attached directly to the box cantilevers, while two more panels at the front and rear, have their free edges supported by catenary edge cables.

Running continuously around the inside edge of the roof is a maintenance walkway. This runs within the 'U' shaped truss of

the main bays and his its own bridge to span between cantilevers at the front of the infill bays. At six locations around the stadium, walkways are provided above the box cantilevers to provide access to the maintenance walkway.

At the ends of the walkway bridges are movement joints to prevent the roof being continuously linked and provide free thermal movement.

The roof bays resist side to side horizontal loads by two means: the main bay cantilevers and tubular truss form a horizontal portal frame; and the infill arches with their diamond pattern rods form a 'soft' bracing panel. The 'softness' derives from the ability of the arches to bow vertically.

1.2.1. Membrane

The membrane is made of PTFE coated woven glass fibre. It is doubly curved and prestressed to remove all bagginess and prevent undue movement under light loads. The prestressing operation also moves the material stress/strain properties to a position where they may be assumed to be repeatable and approximated as linear anisotropic properties.

The behaviour of an anisotropic prestressed membrane, with large Poisson's ratio effects caused by crimp interchange, is very complex. The following paragraphs are therefore only a rough guide as to the behaviour of the membrane.

In the main bays the downwards loads are primarily carried by the fabric spanning from side to side between arches, whilst under upward loads, the curvature betwen arches reverses and the forces are carried both side to side and front to back.

In the infill bays the downwards loads are carried by the membrane spanning between arches, whilst the upwards ones cause the membrane to span side to side between cantilevers.

1.2.2 Braced Arches

The braced arches are pinned at both ends, at the front to the truss, and at the back to the rear beam and bipod structure. The arch itself lies below the fabric, which is attached to its upper surface, and is a stainless steel tube.

The rods are also stainless steel and lie in a vertical plane below the arch. There are six rods, three of which run from one end to points along the arch, with the other three doing the same from the other end.

Under upwards load the arch is in tension and the rods do little except to maintain the arch's shape. Under downwards load the rods prevent any part of the arch from moving upwards and therefore prevent the arch buckling in its own plane, in any of its lower modes, thus reducing its effective length under compression.

1.2.3 Tubular Truss

The truss spanning between the cantilever tips is constructed of fully welded tubular sections and has an open 'U' shaped cross section. The typical trusses are all the same height but reduce in width from bay 1 to bay 6.

Under load the truss carries moments about both principal axes and torsion induced by the thrust from the arches being slightly eccentric to its shear centre.

Under downwards load the top chords of the truss sides are in compression. These members are stabilised laterally by the diagonal members in the vertical sides acting as cantilevers springing from the bottom plane of the truss.

The bottom face of the truss is connected to the cantilevers fully at both the inner and outer chords. this provides a moment connection on plan and allows the truss to act in conjunction with the cantilevers as a portal frame to resist horizontal sway.

The truss supports the walkway, carries the stadium floodlights on its front face, and also supports the roof membrane on the rear chord of the bottom face.

In the untypical bay 7 the truss is triangular in cross

section with a single chord below the front chord of the horizontal face. This truss supports the large scoreboard which is assumed to measure 19 metres long by 9.5 metres high by 2.5 metres deep and to weigh fifty tonnes. The scoreboard is located symmetrically about the lower chord of the truss and there are two tripods which extend downwards to locate the bottom edge of the board.

1.2.4 Box Cantilevers

The cantilevers are fabricated from steel plate welded to form a rectangular box section with overhanging flanges top and bottom. For ease of construction the cantilever is bolted to a much small fabricated 'cantilever foot' which is in turn connected to the concrete tribune structure by large, high tensile rods.

The cantilevers are braced horizontally by the front truss, the rear beam, and the intermediate struts. They also form the legs of the main bay portal frame and the side of the infill braced bay.

1.2.5 Roof Struts

The braced arches are stabilised out of their own planes by stainless steel tubular roof struts. These pass below the arches and are bolted to them by two bolts to provide a moment connection.

1.2.6 Rear Beam and Bipod Structure

At the rear of the main bays is a fully welded tubular steel structure comprising the rear beam and three bipods. The rear beam carries the rear edge of the membrane and a roof gutter. It is in turn supported by the bipods, which also carry the thrust from the braced arches and are bolted to the concrete tribune structure below. The rear beam is attached to the cantilevers by a bolt group which provides a moment connection.

1.2.7 Infill Structure

This is made of stainless steel and consists of four tubular arches, each with four rods. The arches are pinned at both ends to the main box cantilevers and are stabilised, out of their own planes, by the rods which run from the crown of each arch to the cantilevers, midway betwen arches, forming a diamond pattern on plan. The effective length of the arches under compression is taken as the distance between the crown and the springing point.

1.2.8 Walkway bridge

The walkway bridge consists of two rectangular hollow sections as main chords, with associated horizontal cross beams and handrails. Although there is a movement joint at one end to relieve thermal stresses, the walkway bridge will come into play as a strut to prevent excessive distortion of the infill bay, and

ensure that the horizontal frequency of horizontal oscillaltion of the roof is high and cannot be excited by wind.

ARUP Ove Arup & Partners
Calculation Sheet

Job No.	Sheet No	Rev.
15575/05	1·2/5	
Member/Location		
Drg. Ref.		
Made by RLD	Date 16/8/88	Chd.

Title BARI ROOF
DESCRIPTION

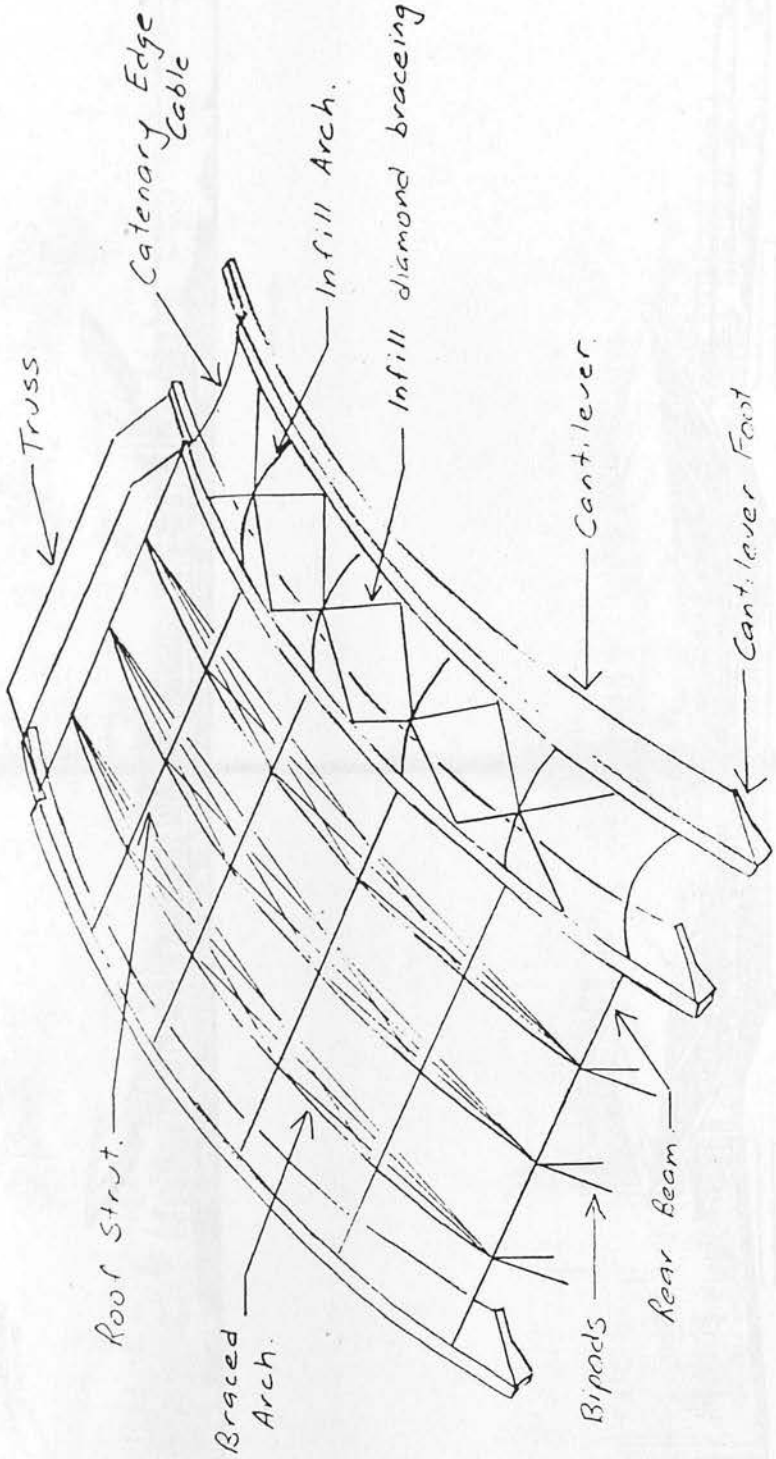
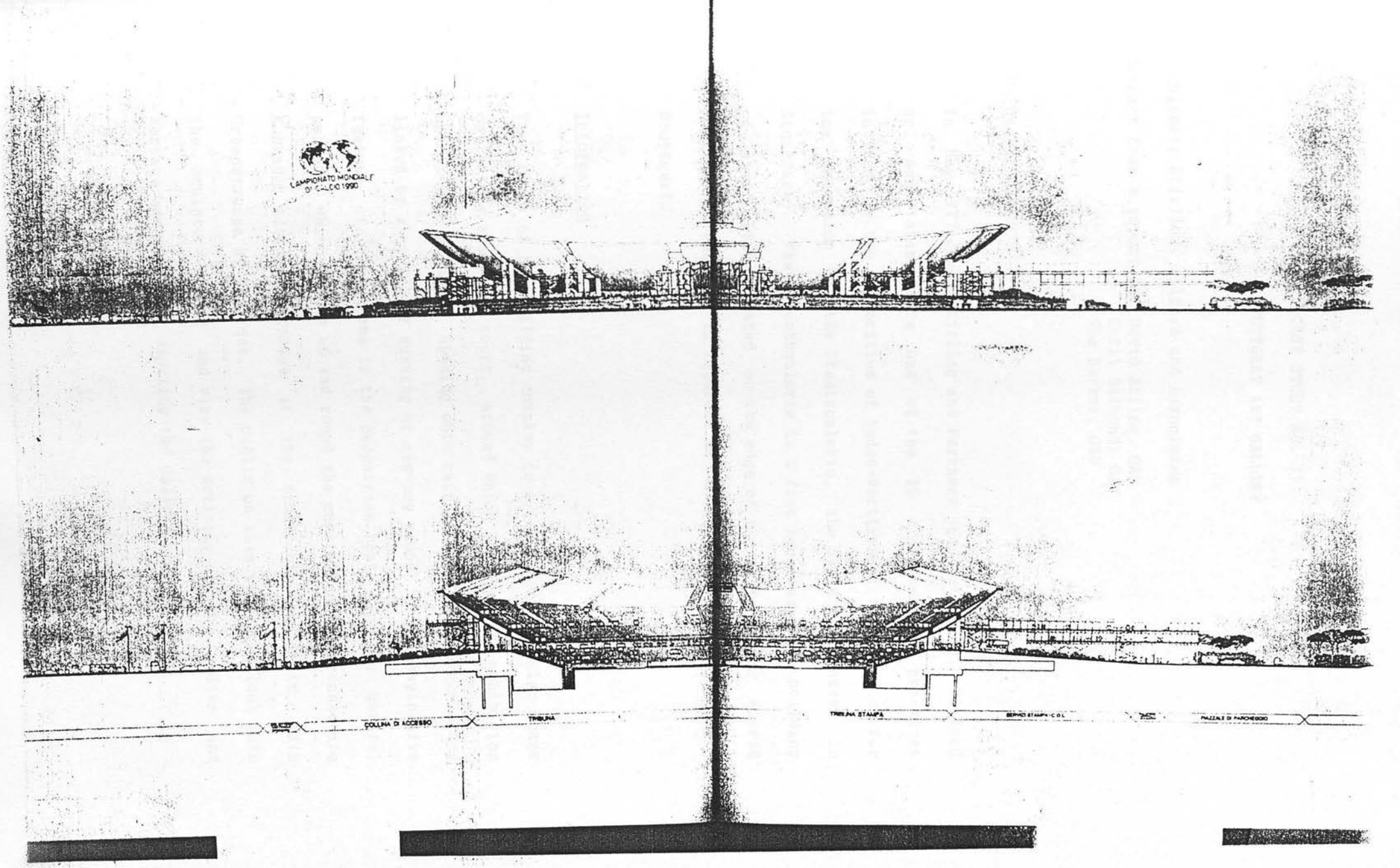


FIGURE 1. SIMPLIFIED SKETCH OF A BAY AND ADJACENT INFILL PANEL


CAMPIONATO MONDIALE
DI CALCIO 1990
COLLINA DI ACCESSO
TRIBUNA
TRIBUNA STAMPA
SERVIZI STAMPA - C.O.L.
PIAZZALE DI PARCHEGGIO

CASE STUDY No. 37

STUTTGART ART GALLERY

Architect: Stirling, Wilford and Associates

Extract from a report by: David Atling, OAP
Cecil Balmond, OAP
Tom Barker, OAP

1.0 Background

In May 1977 James Stirling and Partners (now Stirling, Wilford and Associates) were one of the 10 architectural practices invited by the authorities of Baden-Wurttembert to compete for the extension of the Staatsgalerie, the State Art Museum in Stuttgart. The Straatsgalerie is a fine neo-classical building of about 1825 located on the edge of the city along Konrad Adenauer Strasse. It is regarded as one of the major landmarks of Stuttgart.

2.0 Information

The focus of the building complex is a large open circular space called the sculpture court, around which the various exhibition spaces are planned, opening onto terrace and foyer levels, and linked by a series of curving or zig-zag ramps. An imaginative feature of the scheme is the pedestrian route for the general public, which winds up and round the central court, connecting Konrad Adenauer Strasse at the front of the site with Urbanstrasse at the rear. The public on this route can look into the sculpture court and view the activity of the podium and terrace areas without visiting the gallery.

Local marble and sandstone are used extensively for the external cladding to the buildings. Contrasting with this traditional look are the twisting glass walls of the foyer, the steel lattice canopy structures and the angular lines of the elevations themselves; it is a striking mix of high-tech and classical forms.

Apart from the art gallery, there is a theatre, a music school and a library in the building complex. The total cost of the project is estimated at just over 82m Deutsche Marks.

3.0 The Site

The site (fig. 1) which was largely waste land, is approximately 140m long x 90 m wide, bounding by the old gallery to the north, by the dual carriage to the west and minor roads to the south and east. The level difference from west to east, this is from front to back. is 15m.

Wells were set up on site to check ground water levels and initially these wells indicated that the major part of the site excavation would be above the water table. But the water in the wells continued to rise, not only in level but in temperature; and a hot spring was discovered. As a result the building was lifted slightly and special precautions were advised for watertightness of the basement and for the routing and protection of subsoil drainage to avoid contamination of the hot springs.

4.0 Concept Stage

The initial task for the design team was to work out a structural

and services concept that would fit the tight planning of the various levels and yet not raise the building height above that of the existing gallery. Keeping the basement excavation to the minimum, and out of the water table preferably, was an added constraint to vertical storey heights.

For the structure, downstand beams were avoided where possible. Floor slabs were designed to span directly onto columns or to be supported by walls serving as full storey-height beams. In Germany, reinforced concrete wall construction was priced more cheaply than column infill block construction, so the use of walls as beams became feasible. This was also compatible with the planning of the internal spaces. Flat slabs supported by these storey-height wall beams kept the horizontal structural depths to a minimum.

The use of walls, however, restrained the structure to the extent that the conventional approach to expansion joints could not be adopted. The 102 m x 90 m gallery structure, being exposed externally to temperature range of +/-20 degrees C and internally to an air conditioned environment, would normally have needed more than one break line in the structure, as early sketch proposals indicated. However, calculations showed that the extra cost of reinforcement was about 4% for having no joints in the structure. It also meant that the possibility of water leaking through joints in the structure and damaging valuable paintings was thus eliminated. The theatre structure, 37m x 65 m, was isolated from the gallery structure by an expansion joint.

The mechanical system chosen was an all-air system with low

velocity distribution. The layout of the services was planned to avoid large ducts running horizontally which would have raised the height of the building. Consequently, the plantroom was stretched out along the whole length of the building with vertical service risers connected directly to strategic areas.

The plantroom is located adjacent to the car park. Excavation and retaining wall costs were reduced by pulling the car parking and plant space as far forward as possible (Fig. 2).

Above this level are the main features spaces of the foyer, the temporary exhibition room, the large drum of the sculpture court, the lecture theatre, and the drama theatre foyer. The permanent exhibition spaces are on the uppermost level, opening onto terrace areas. At this level there is the theatre and rehearsal room (Figs 3 and 4). It was initially intended to provide a fully glazed roof over the upper gallery areas to maximize natural light conditions for viewing the pictures. However subsequent energy considerations caused the area to be reduced and the roof void was made 2m deep with steel trusses spanning onto the gallery walls, with the ductwork hugging the walls to avoid reduction of daylight.

For maintenance of the ceilings and the daylight control louvres in the roof spaces, catwalks and permanent moveable trolleys were provided within the roof zone, integrated into the structure and services planning concept.

5.0 Lighting

The competition brief required maximum use of natural light for viewing exhibits. Initially the upper gallery rooms were planned with fully glazed roofs. As the design progressed, however, areas of glazing were reduced to lower the air-conditioning load and attendant running costs, but still allowing viewing under natural light for 76% of normal opening hours.

For viewing water colours and oil painting the illumination levels of 50 and 200 lu respectively were specified on the vertical surface 1.5m from the floors. With artificial lighting these levels are achieved by switching circuits in the upper gallery room and by dimming in the temporary exhibition space.

Natural light is controlled by adjustable motorized louvres mounted in the ceiling void. Initially these louvres were planned to operate automatically vis individual room sensors. But in the end the client required them to be under manual control of the guide responsible for each room. The substantial nature of these louvres help them to act as anti-burglar devices and also as thermal insulation when fully closed during the winter hours of darkness. The louvres are located beneath the roof glazing and span onto the top booms of the structural trusses.

Along the bottom boom of the trusses is a steel grillage to support the ceiling layer which is made up of glass incorporating an ultra violet filter, which prevents harmful radiation entering the exhibition space. (Fig 7).

Extensive tests were carried out with paintings hung in the model room for daylight and artificial light, using various glazing solutions for the roof and ceiling.

It was found that the sole use of float glass produced an unacceptable green hue to exhibits. This was overcome by incorporating Albarino glass into system.

Albarino glass has been developed especially for use on solar cells and has very high transmission factors for all wavelengths of light. But expense limits the extent of its use.

Structure

From the outset the main problem to contend with on the structural design of the gallery was the large span of the roofs of the lecture theatre and changing exhibition rooms. The span was approximately 21 m and the available depth for structure and services was in the order of 1.2 m. The upper galleries and terrace areas had to be carried by these slabs and various grid solutions were tried, from the straight, deep rectangular coffer grid to other configurations that reflected the distribution of bending and twisting moments in the structure more accurately.

As expected, computer analysis gave high torsion stresses in the grillages. But in collaboration with our German colleagues we agreed that the high torsion stresses would not really arise in the actual structure to the extent indicated by theoretical analysis and the walls of the upper galleries could be located on plan with more freedom, and minimizing the slab depth with the

use of the large columns heads also improved the available roof void depth at the higher levels allowing for better access and better integration of the structure and services.

During early concept stage the other matter for debate on the structure was whether an expansion joint was desirable or not. Wherever a joint line could be drawn the details became extremely contorted. The tight 'layering' of the various levels did not lend itself to independent structural units; the walls located along all external sides of the building removed any 'give' in the structure at the edges normally free for expansion and contraction, therefore it had to be a 90 m x 100 m structure without joints. For a while prestress was considered but it was agreed then to abandon a sophisticated treatment of the expansion problem problem and to deal with it by simple crack control procedures, based on the even distribution of strains induced by temperature and shrinkage.

Fig. 1
Site plan

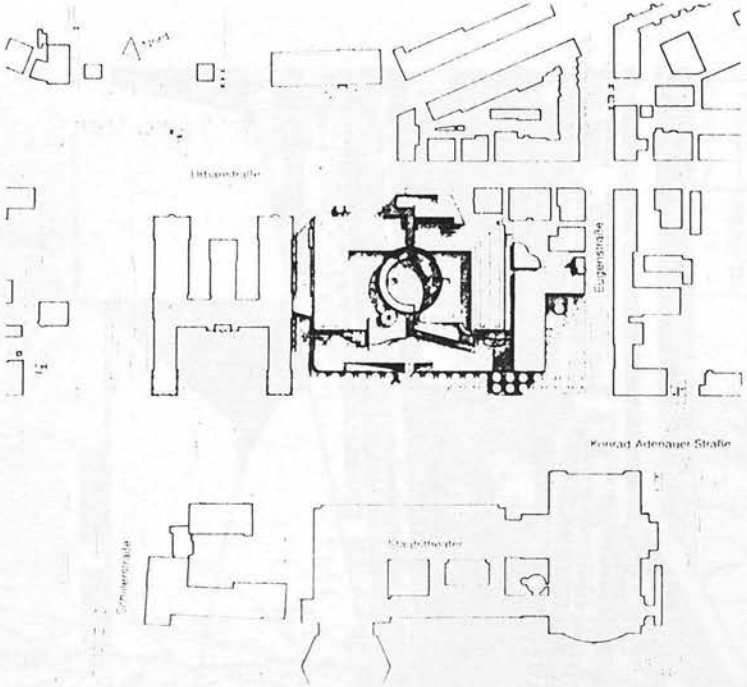


Fig. 2
Plan at car park and plantroom level

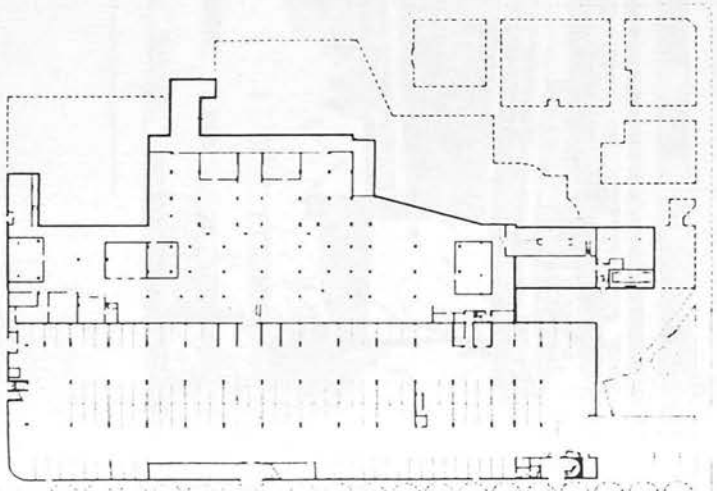


Fig. 3
Plan at entrance level and east-west section

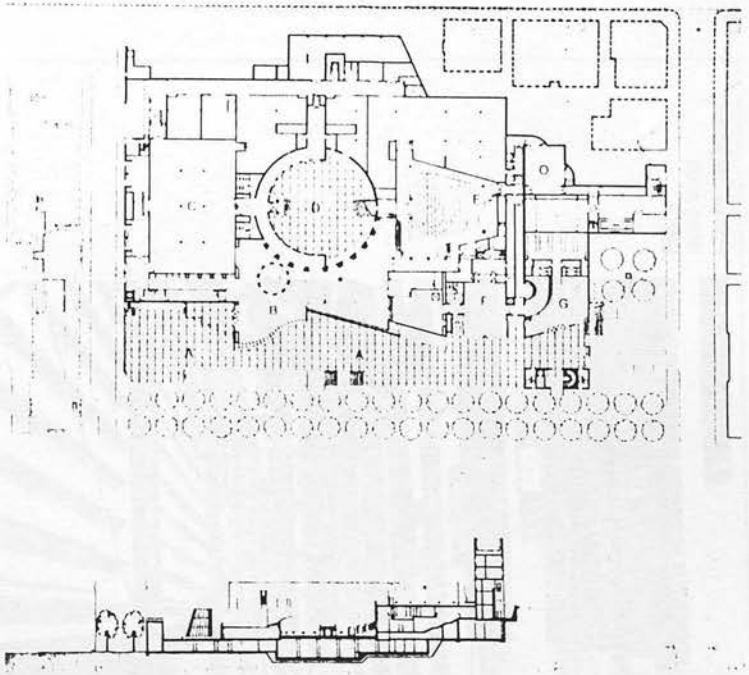
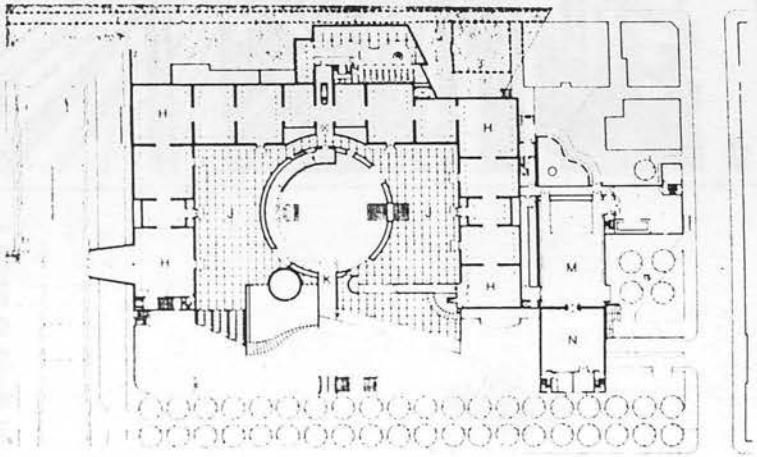


Fig. 4
Plan at Gallery level and north-west section



6
ation from Konrad-
nauer-Strasse

ting sources

erally lighting sources within the
elopment are fluorescent, though in the
rance and reception areas it was intend-
to use incandescent lamps to create a
m atmosphere.

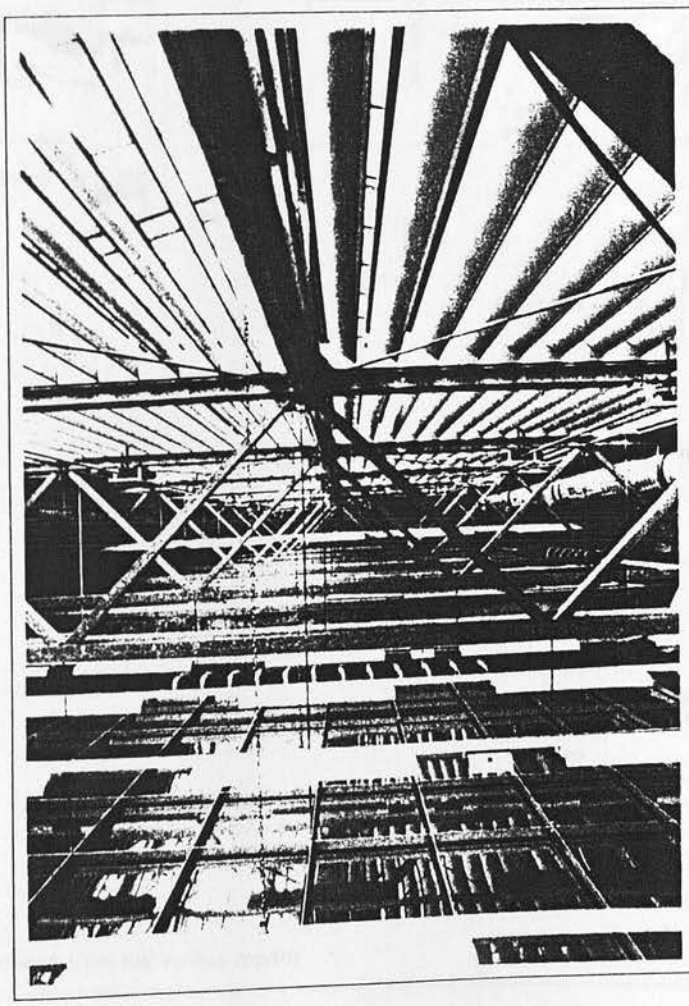
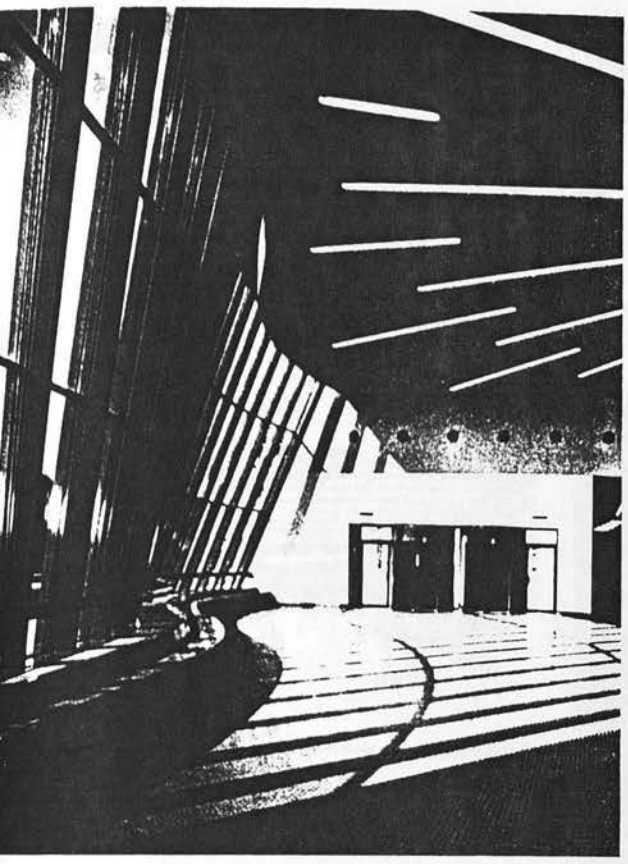


Fig. 7
Roof void
above galleries



g. 8
trance hall

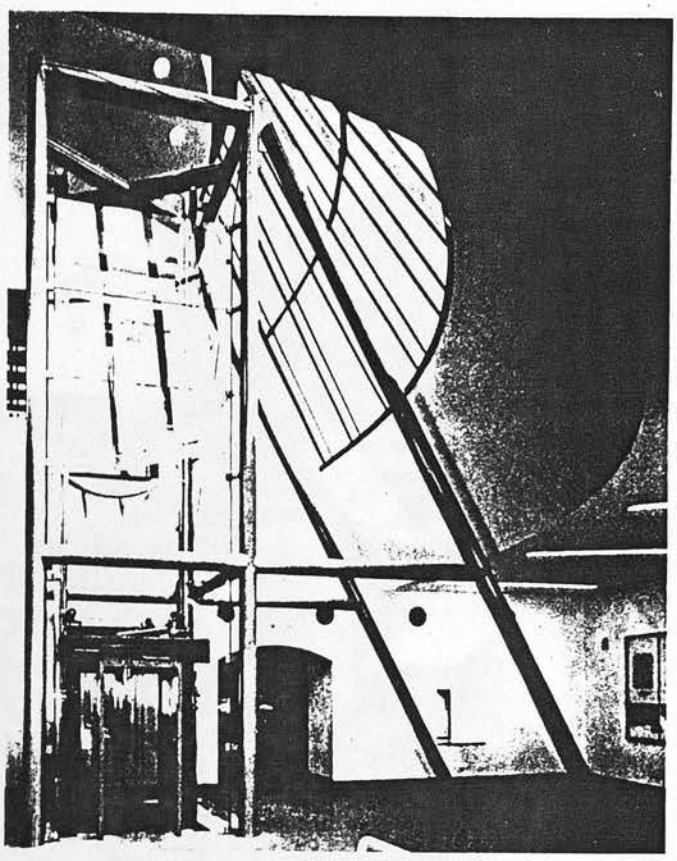


Fig. 10
Gallery lift

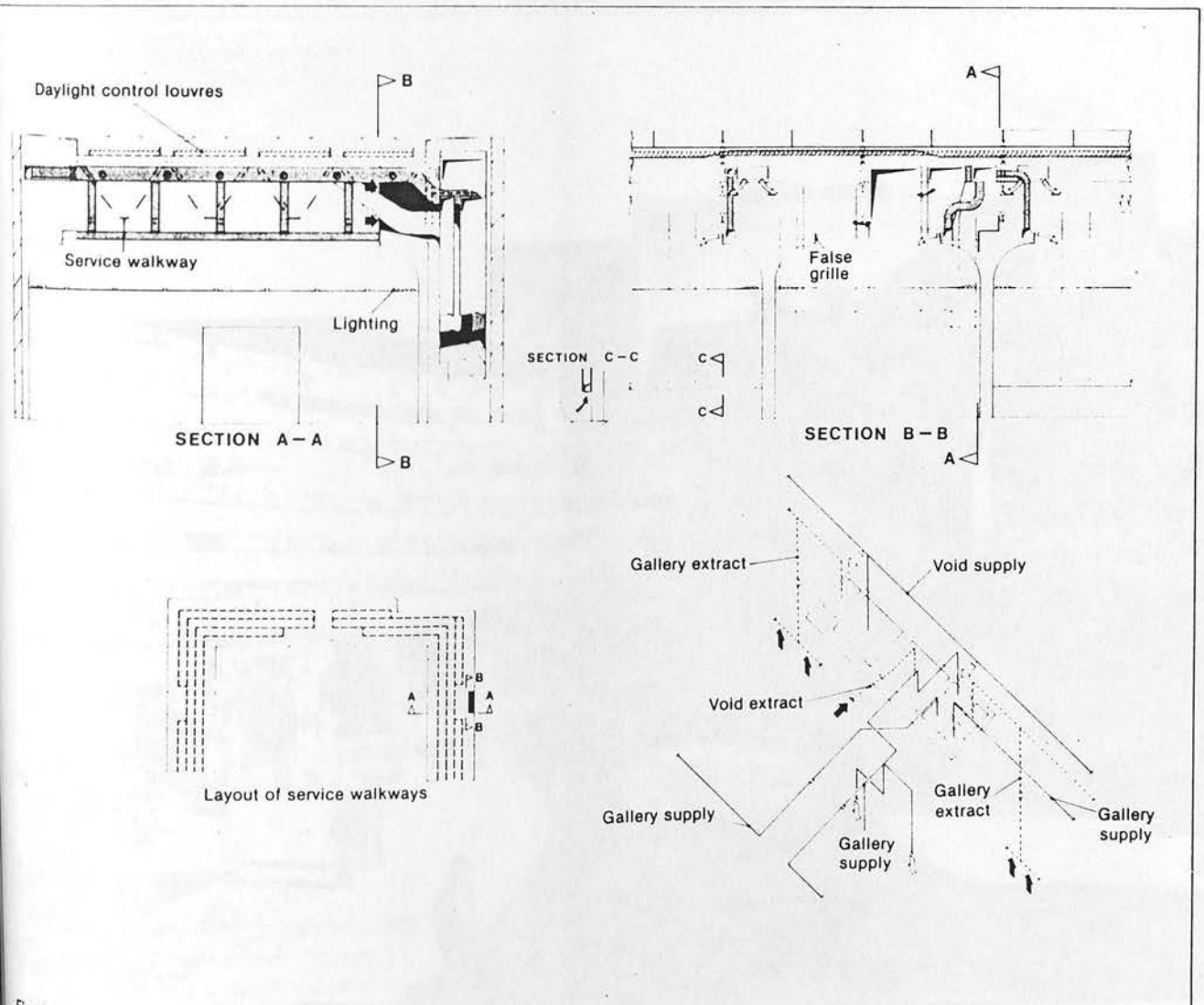
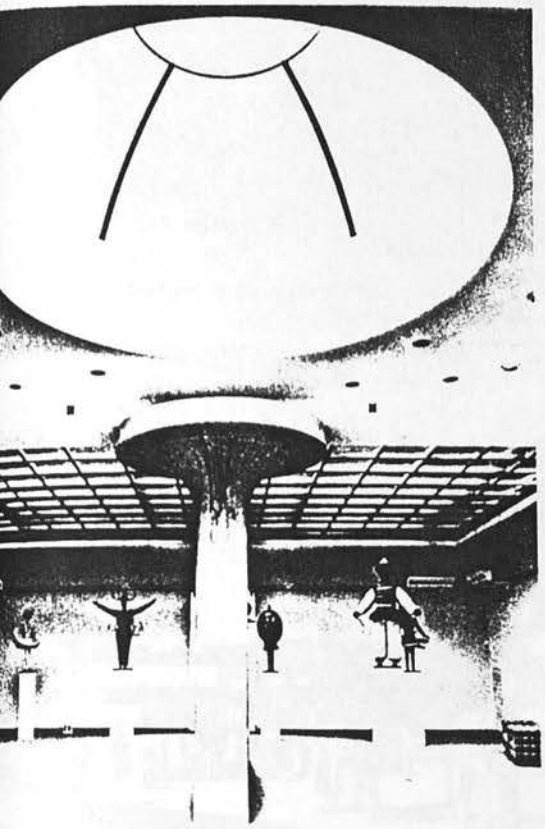


Fig. 9
Detail of services at Gallery level (Reproduced from the Vu Bau report)



11
ery room from link bridge to old building

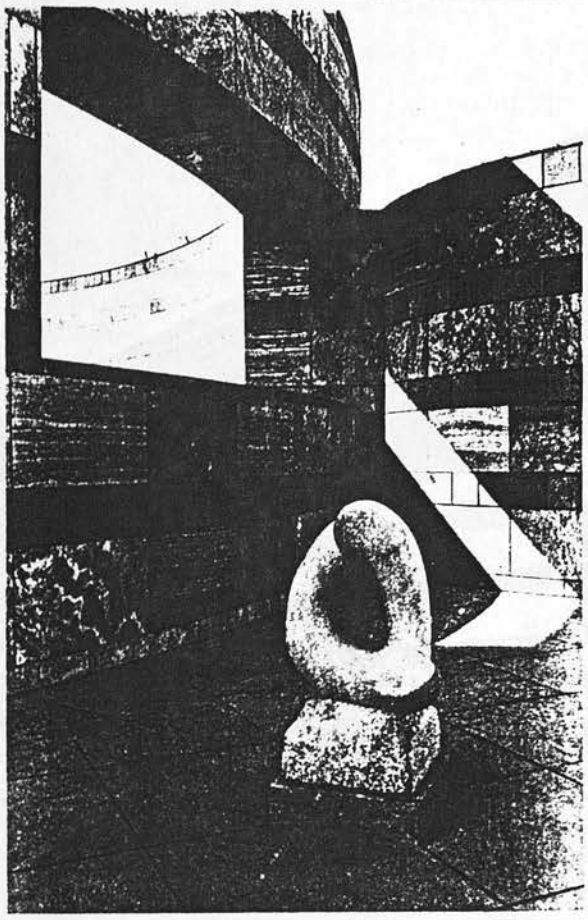
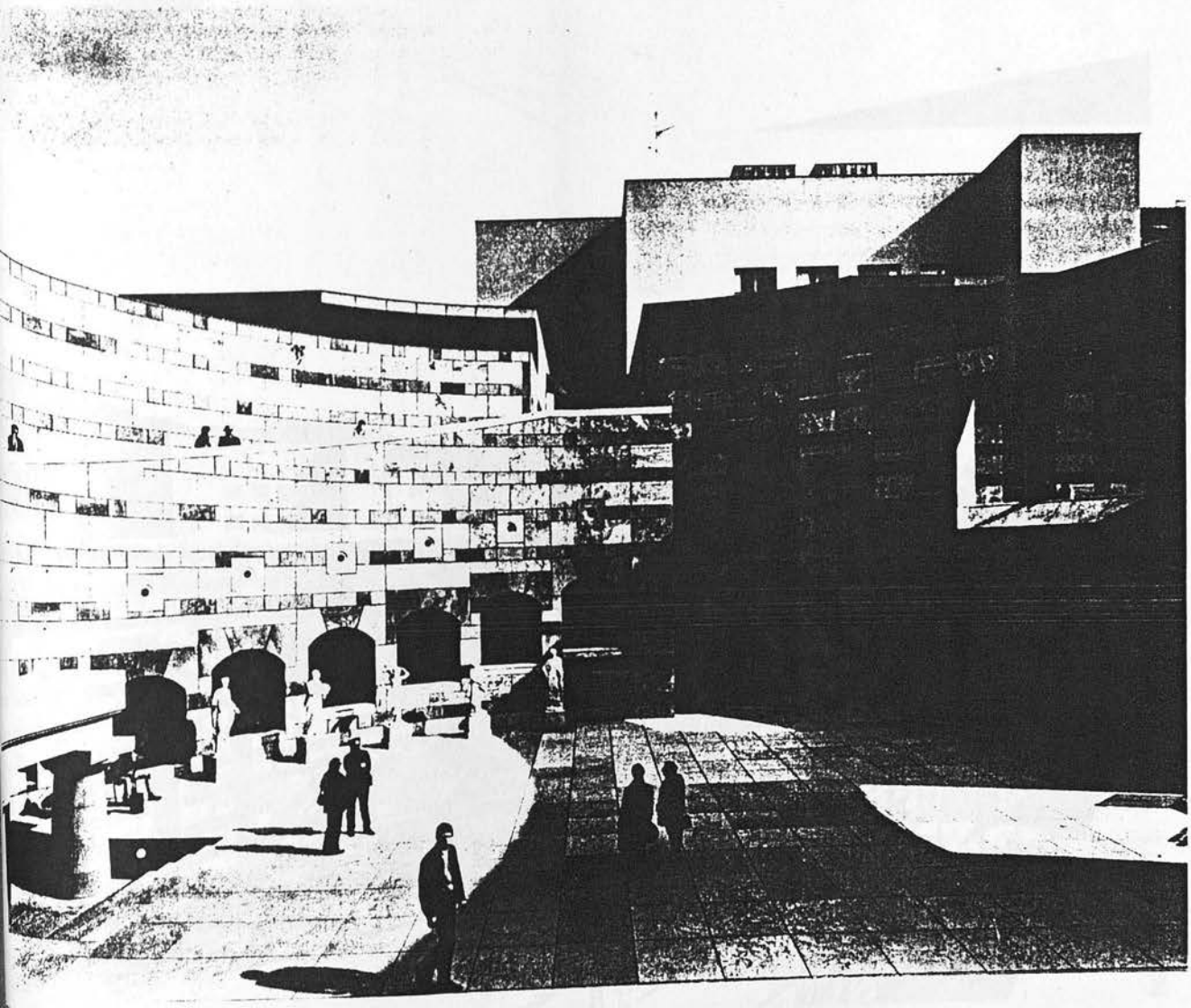
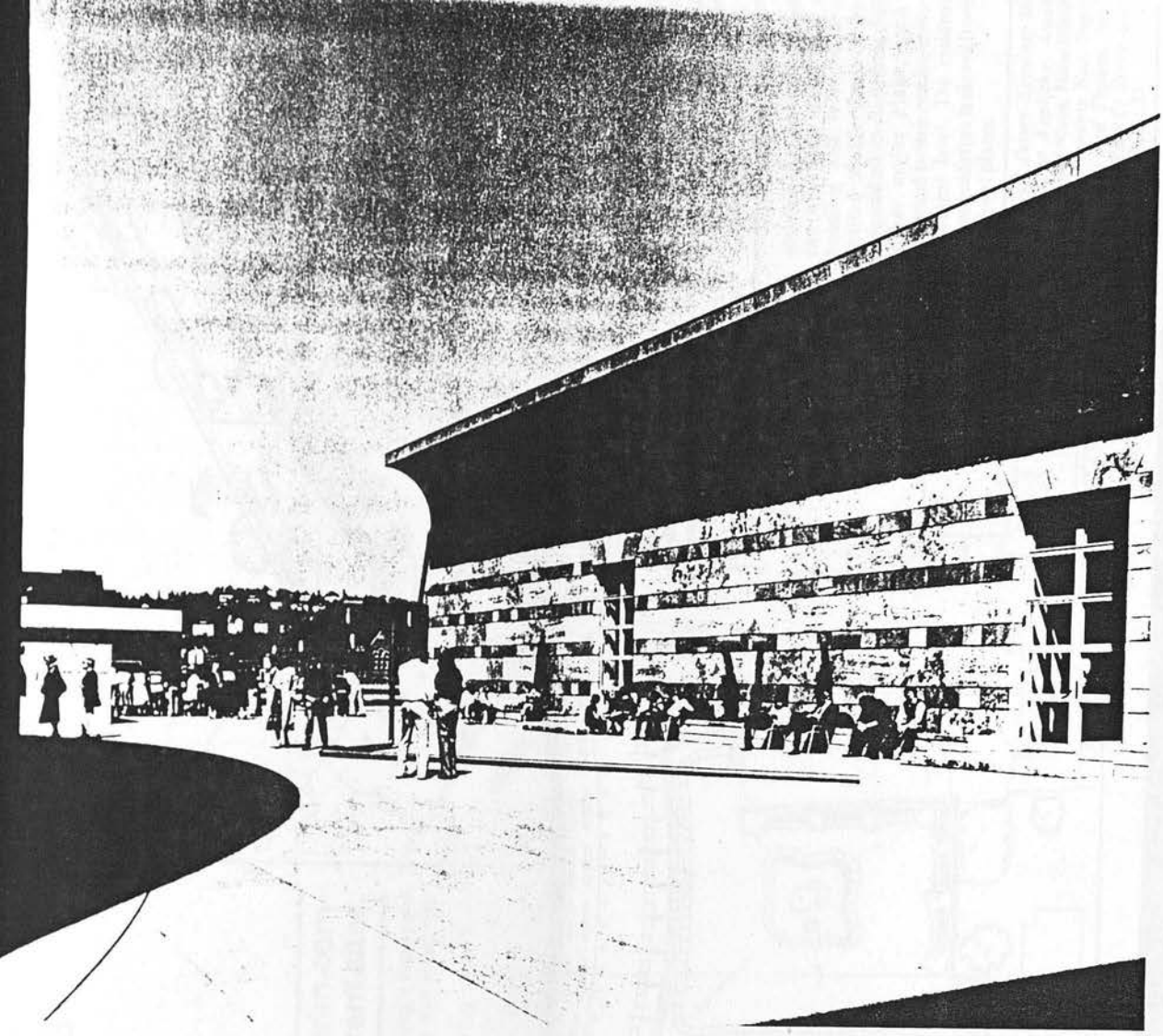
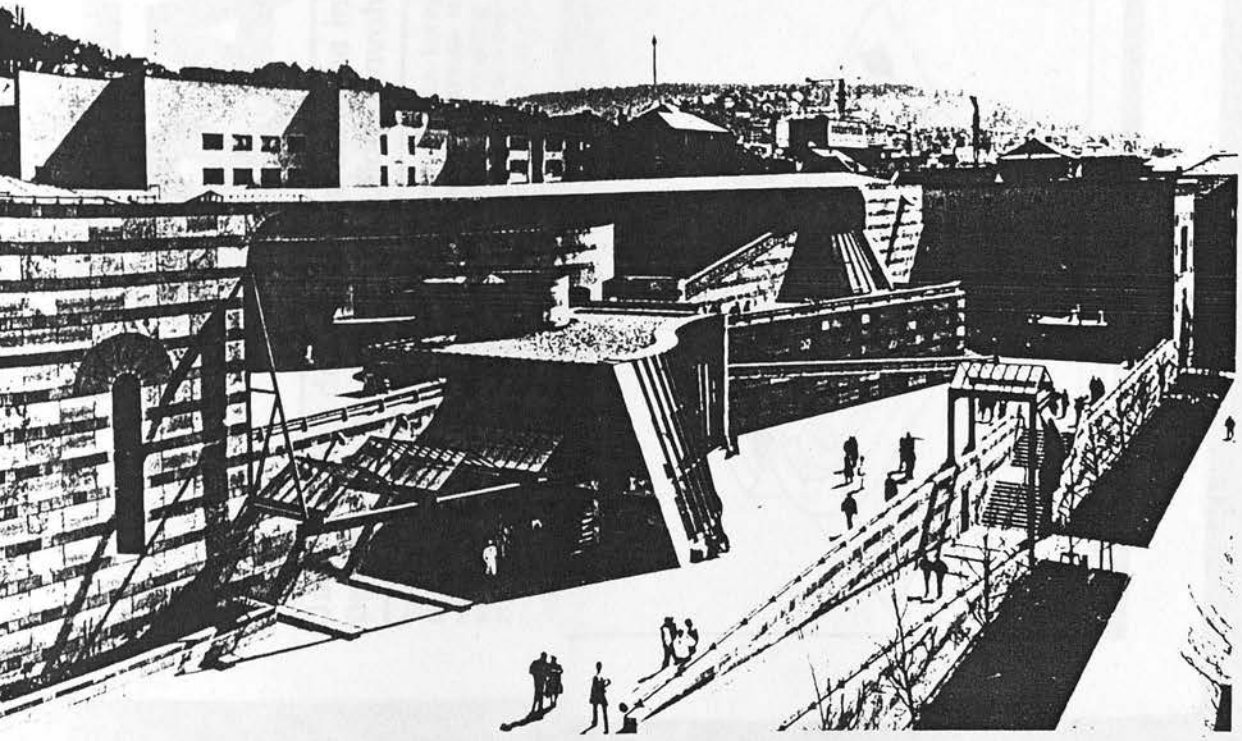


Fig. 12
Sculpture



g. 5
ulpture





BERLIN WRAP

case study 38

Jan Kaplicky designed this scheme in the recent Berlin competition for an industrial building involving four different sites.

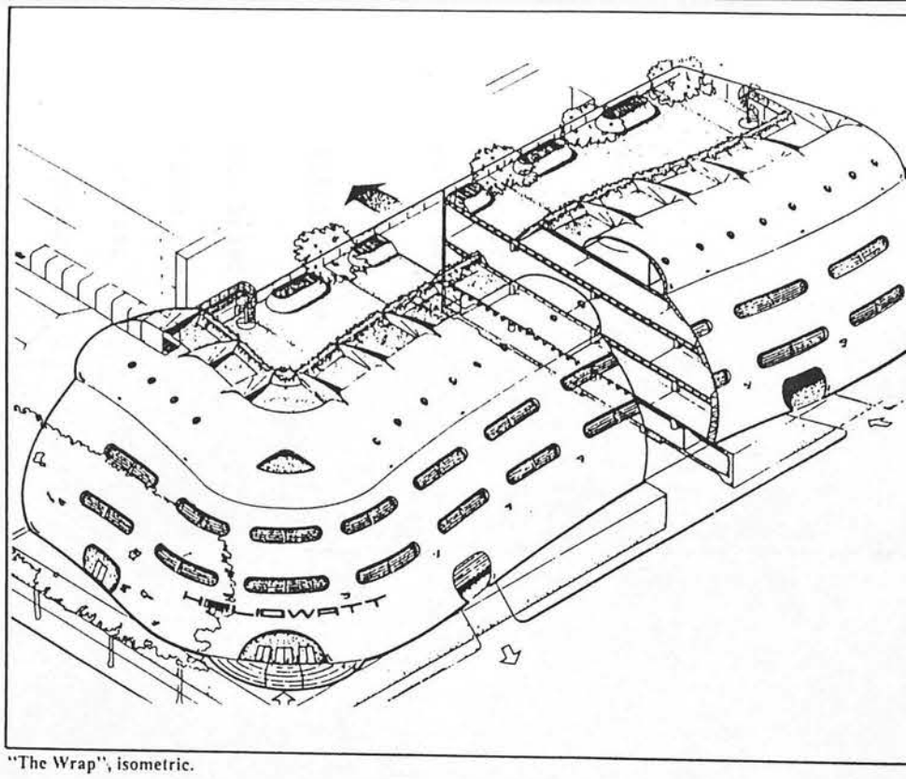
REDEVELOPING a part of the block off Bismark Strasse is a unique opportunity to create a new image and a higher profile for Heliowatt. The title "The Wrap" is derived from the notion of wrapping a piece of fabric around a contoured frame, like a piece of clothing draped over a body or the wraparound windows and cladding of the 1920s and 30s.

"The Wrap" is a radically different aesthetic from the familiar Berlin architecture. Electric blue is not the local vernacular colour but the building is integrated into the city fabric not by its similarities, but by its differences. A new contour to the site is formed by wrapping the shiny metallic blue skin over the structure and against the line of the street.

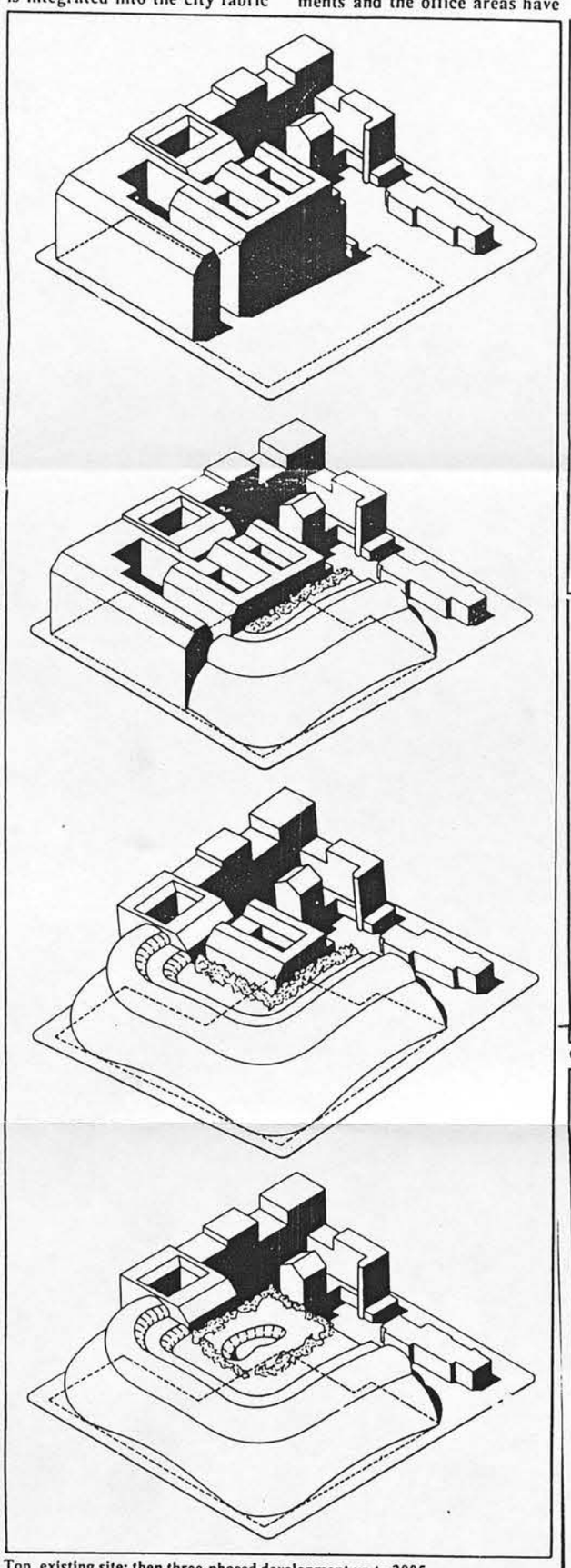
The activities and functions in the Heliowatt building are arranged as a horizontal slice, rather like a sachertorte. There is no hierarchy of importance or prestige within any of the activities, but the three-dimensional shiny blue skin wraps over all the layers including the service and car park access. Both the apartments and the office areas have terraces overlooking a courtyard so that everyone can enjoy the private garden.

To concept of "the wrap" is to provide flexible spaces and areas which can be expanded and rearranged to meet the fast changing demands and technological advances in production.

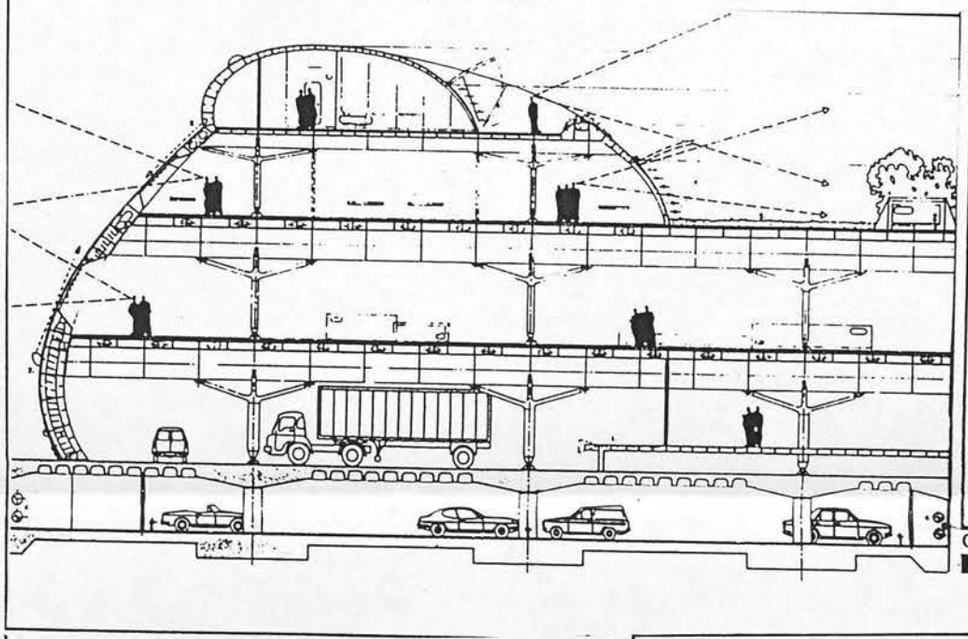
The overall strategy for the site is one of phasing. At the same time the expression of the building is deliberately strong so that it can stand alone at the completion of any stage. On completion of the last phase, the production area also opens out onto the garden, allowing everyone to benefit from the outside space.



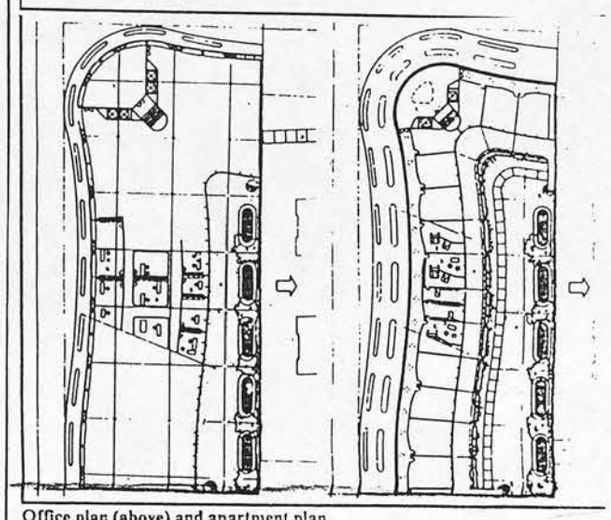
"The Wrap", isometric.



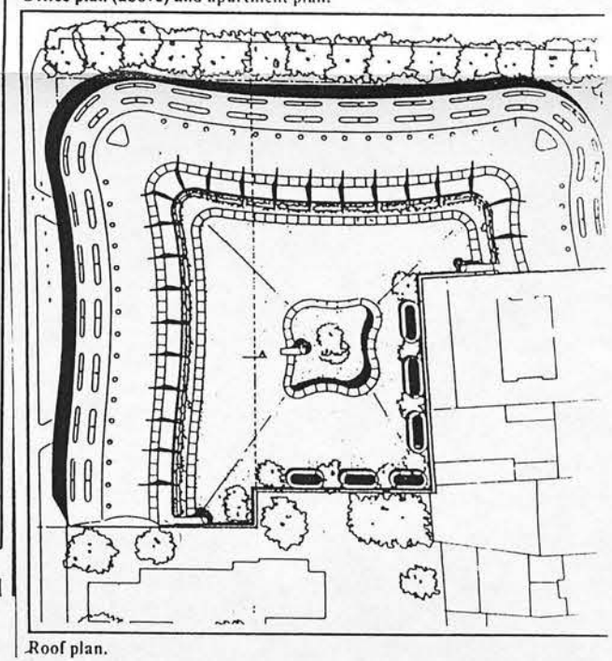
Top, existing site; then three-phased development up to 2005.



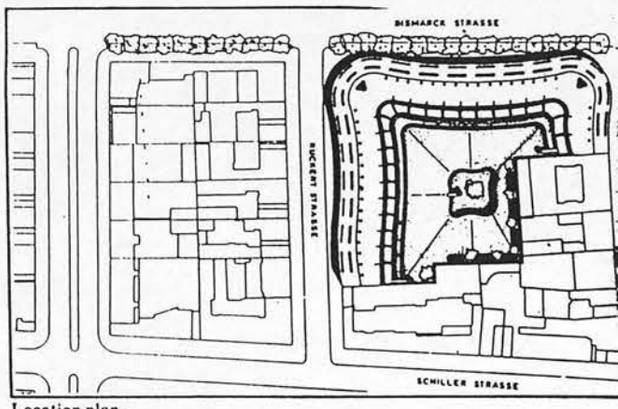
Concept sketches.



Office plan (above) and apartment plan.



Roof plan.



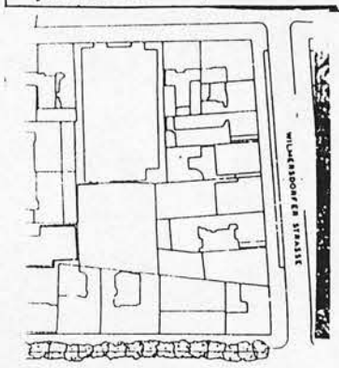
Location plan.

The superstructure for the main building comprises a lightweight concrete floor in permanent formwork which acts compositely as secondary steel beams, spanning onto a primary grid designed to radiate following the two corners in the building.

The primary beams are tapered open sections supported at intermediate points by Y-shaped columns, reducing overall spans and concrete filled for fire protection. Primary beams also accommodate supply and exhaust ducts.

The sloped semi-monocoque skin with ceramic finish acts as perimeter support to the lower floors, with the skin continuing over, forming not only the walls but also the roof. The basement and car park are formed utilizing a similar structural principal but substituting an in-situ reinforced concrete frame and slab.

The building is air-conditioned by a variable volume system supplied by central air-handling plant in the basement. Primary supply and exhaust ducts are distributed within the thickness the external wall. The ai handling unit is supplied wi chilled water from a roo mounted air-cooled water chill Other major plant, such electrical switch rooms, is also basement level. The residenti accommodation is served from i own local plants.



Architects: Future Systems; Ma Collins, Jan Kaplicky, Aman Levete, David Nixon, Fiona Whall Structural engineers: Hunt Projec Tony Hunt, Neil Thomas. Serv engineers: YRME; Steve Gre Modelmakers: Unit 23

CASE STUDY No. 39

BLACKHEATH MEETING HOUSE

Extracts from a report by: Edmund Happold and
Ian Liddell

Architect: Trevor Dannatt

Engineers: Happold, Liddell

1.0 Introduction

The brief was to provide a hall to seat 100 people, two committee rooms, a kitchen, lavatory accommodation and a link to the existing church hall at ground level. (Fig 1) A town planning requirement was to allow space for parking four cars and this, together with the considerable change in level between Lawn Terrace and the site ground level, made it sensible for the pedestrian entrance to be off Lawn Terrace at the upper level into an entrace hall and tea kitchen at the Meeting Hall, Stairs then lead down to the ancillary accommodation, the car parking and the link to the church hall. (Fig 2, 3)

2.0 Architectural Concept

The architect considered a square plan for the main space parallel with the church hall but decided to rotate it 45 degrees to avoid a feeling of congestion aganst the existing building and to isolate the new building. (Fig 4, 9) From this originated the entrance hall, an important space in the social life of the Meeting, with stepped walls in plan, a stepped ceiling and leading to the calm space of the hall. At a meeting; people sat round in a square and natural light from above, provided by a

central lantern, was felt to be appropriate. The corners of the room are cut off space between roof and outside wall which is glazed to provide natural lighting on the wall surface. The walls at the corners are carried up externally as turrets to receive the top of the glazing.

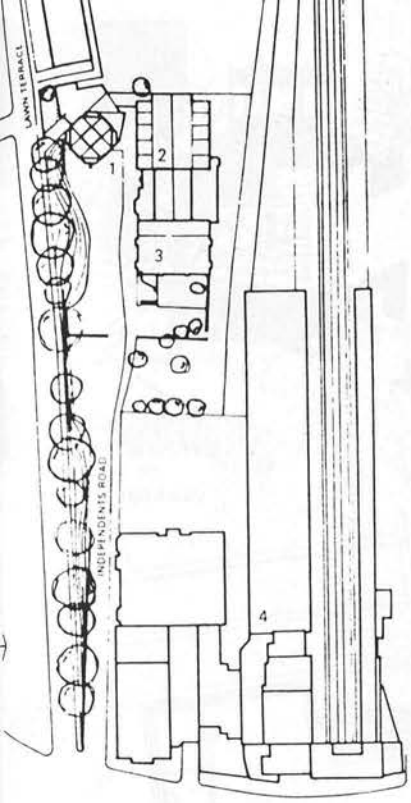
3.0 Structural concept

Structurally, Blackheath Meeting House is similar in concept to Bootham School, York. The walls of the hall act as beams to support the floor and simple deep trusses span onto the walls to support the roof. (Fig 8) At Blackheath the span of the roof was such that the trusses could be made of timber for the compression and bending members with steel ties. (Fig 7) It was felt that this form of construction gave a suitable internal appearance and could be built with a few special joints by carpenters.

Two trusses span in two directions intersecting at four vertical members which extend above the roof level to support the lantern. the joints are formed with steel brackets which bolt to the vertical member and have a vertical tongue. The timber members are formed from a pair of 250 x 50 mm timbers which bolt onto the tongue with shear plate connectors forming a pinned joint.

Under symmetrical loads this arrangement is stable without cross-ties in the central bays and Trevor Dannatt wished to avoid these. For unsymmetrical loads it is a mechanism with one degree of freedom which can be adequately restrained by horizontal cross-ties at the level of the truss compression members. The walls of the Meeting House are designed as concrete beams to

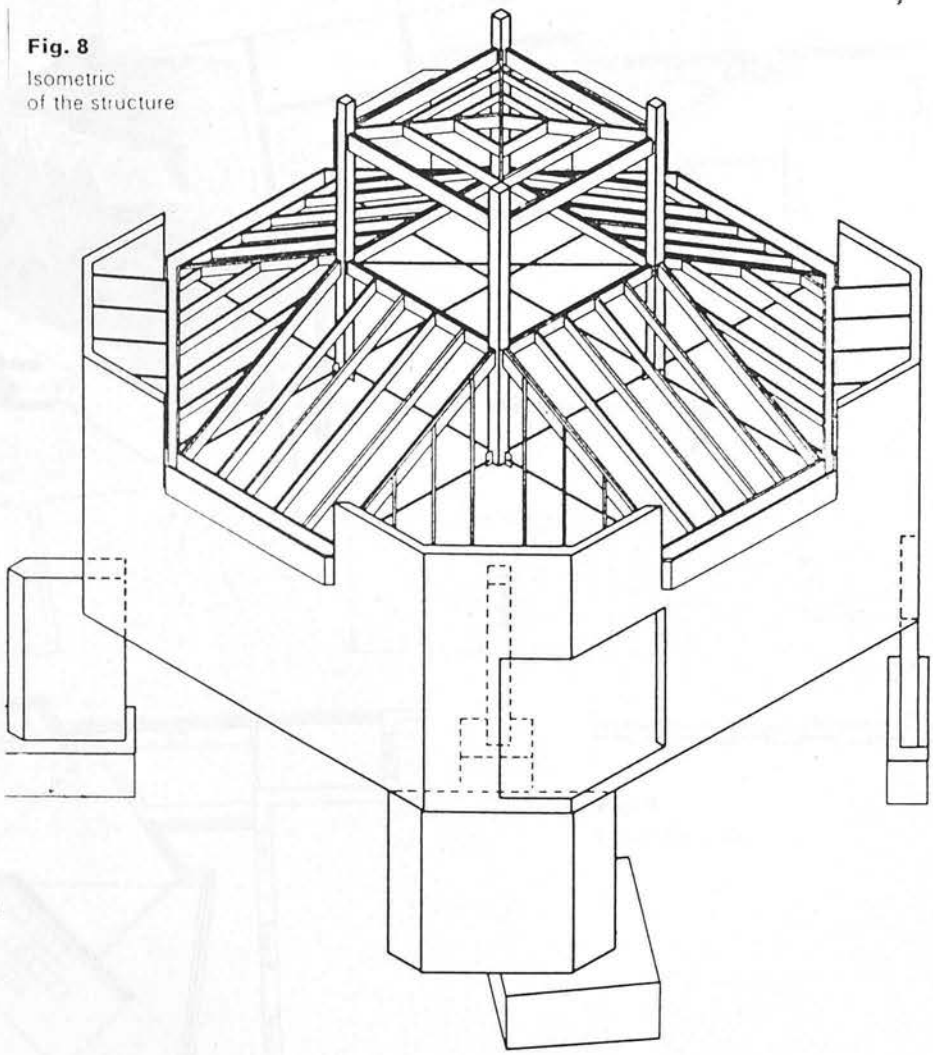
reduce the floor depth and to carry the load from the roof to the four corner columns. They are exposed externally with a boarded finish. A column stands under the centre of the floor to reduce the span of the flat slab. (Fig 4) To allow for existing services and basements the foundations had to be offset under two of the columns, but the 'box' design accommodates the additional forces produced.



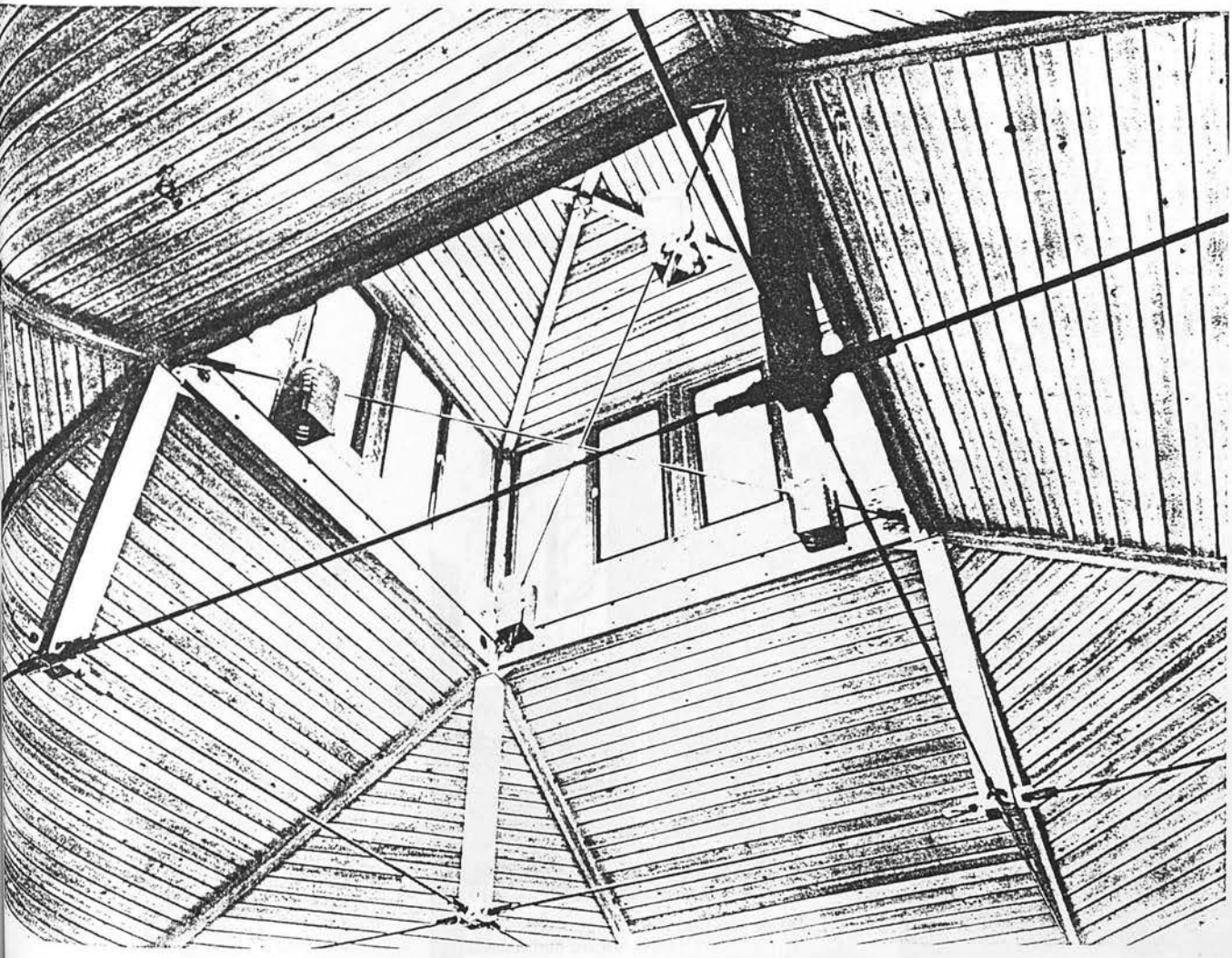
y
Meeting House
Church Hall
Congregational Church
Station

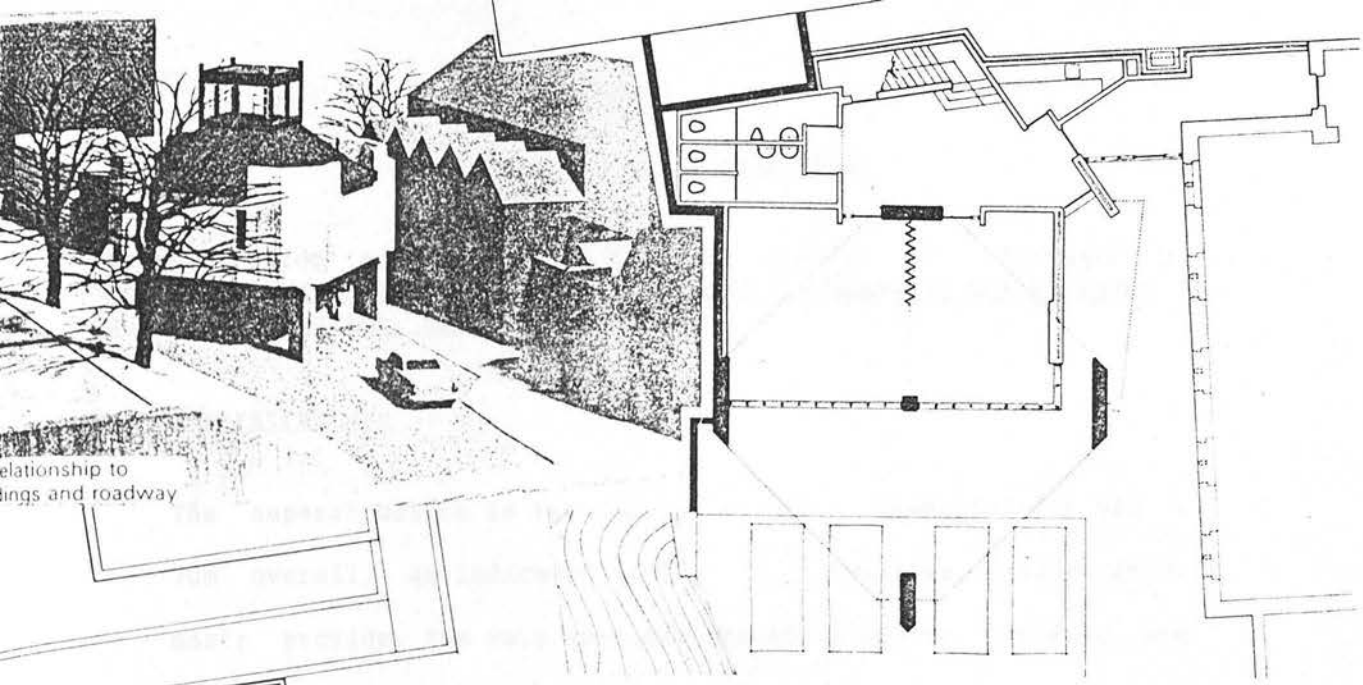
Fig. 1
Location plan

Fig. 8
Isometric
of the structure



ig. 7
oof structure showing lantern and
eel ties





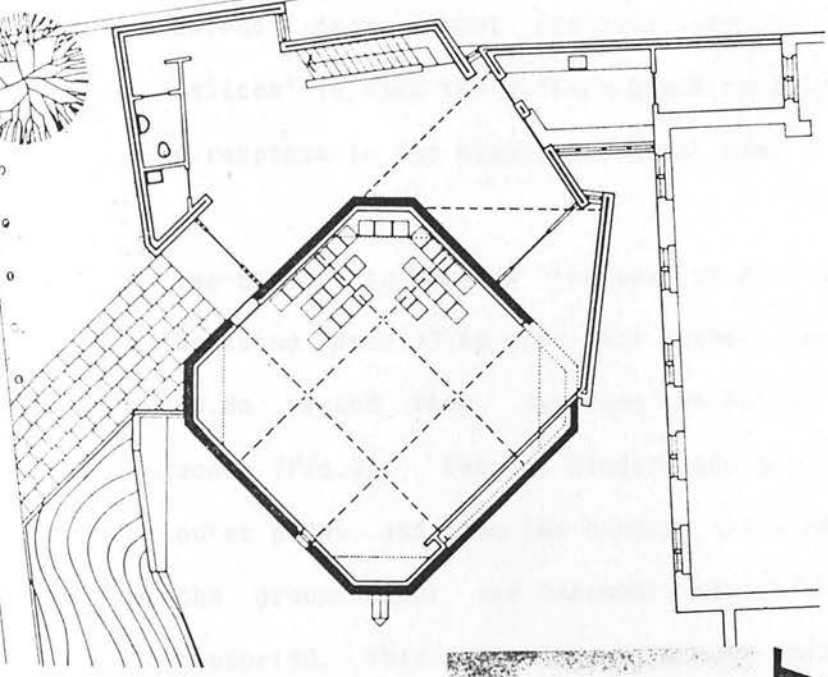


Fig. 4
Lower floor plan

or plan

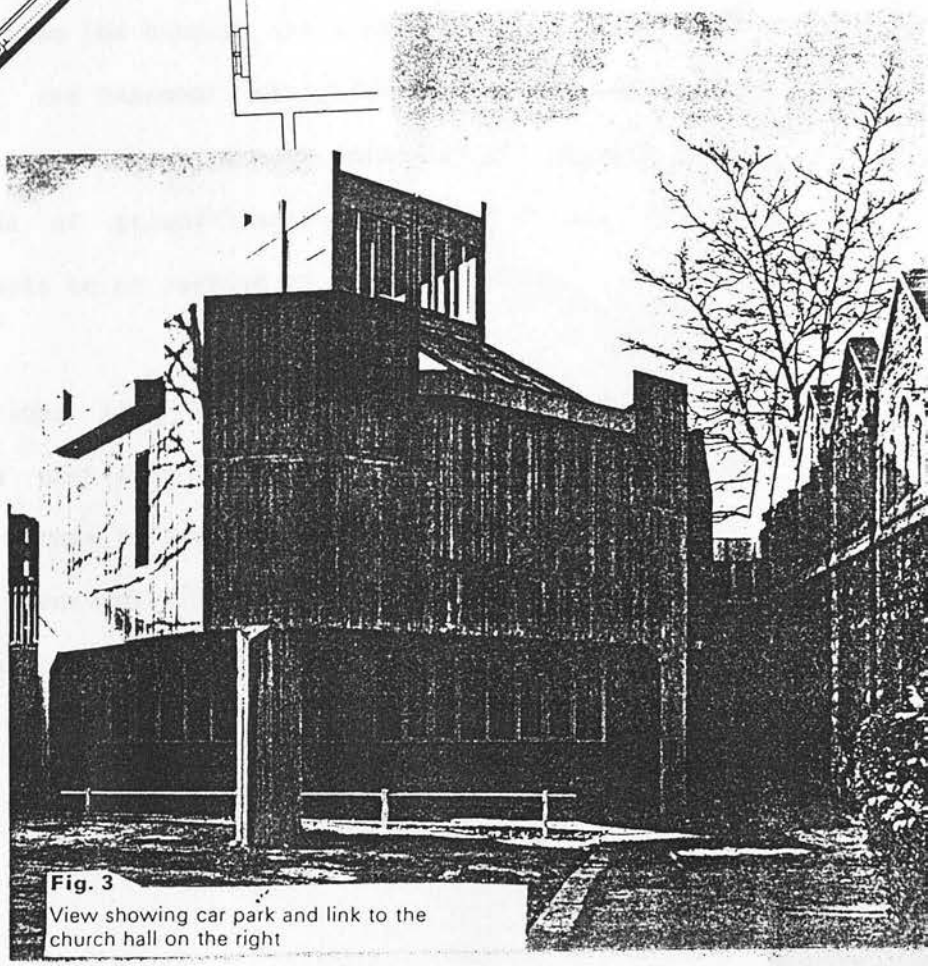


Fig. 3
View showing car park and link to the church hall on the right

CASE STUDY 40

The Hongkong and Shanghai Bank

Excerpts from a paper by J.Zunz, G Glover, A. Fitzpatrick. "The Structure of the new headquarters for the Hongkong and Shanghai Banking Corporation, Hong Kong

1.0 Superstructure

The superstructure is rectangular in plan, approximately 54m x 70m overall, as indicated in Fig. 1. Two rows of four steel masts provide the main vertical structure of the building and extend from lowest basement level to the tops of the three 'slices' to form the cutback geometry (Fig. 5), which itself was a response to the planning regulations.

Two-storey suspension trusses at five discrete levels of the building span 33.6m east-west between the masts and cantilever 10.8m beyond them, dividing the building vertically into five zones (Fig.4). Tubular hangers are connected to the central and outer nodes, and from the hangers are suspended the floors. With the ground-floor and basement structure being conventionally supported, this suspension structure arrangement creates a large column-free zone at ground-floor level, with all of the superstructure loads being carried by the eight masts.

Planning regulations also required that the massing of the building on the east side be reduced. This was achieved by setting back the floors between the masts on that side, the set-back increasing progressively up the building. The resulting floor plans at various levels are shown in Figs. 7 to 9.

The major potential for future increase in the area of the building lies in these east-side setbacks. The adjacent primary beams, plus the masts and foundations, were designed to carry the load from the total infill of this side of the building, thereby making the structure essentially similar to the west side.

The two-storey spaces occupied by the suspension trusses become the focal point for each bank of floors. They contain the reception, conference, and dining areas, and lead onto open terraces which form both refuge areas in the event of fire and recreation areas in normal times, with some of the best views in Hong Kong as a bonus. The structure has been designed to carry the load from mezzanine floors should these be introduced at a later date.

The layout on all floors is essentially the same, being dominated by the large central open space to provide maximum flexibility in layout and operational use. Services, lifts, and stairs, are located on the east and west sides, with the lifts above level 13 serving only the double-height floors. Travel between these levels is by escalator, resulting in faster average journey times and a greater feeling of 'openness' and communication between floors. The central slice of floor in the lowest superstructure zone is omitted to form an atrium between levels 3 and 13 (Fig. 6), around which are located the public access banking areas with a new double-height Banking Hall being at level 3.

A glazed curved soffit seals the base of the atrium at level 3 affording a clear view into the building above from the plaza floor, which forms an a-grade public thoroughfare linking two of

Hong Kong's major pedestrian routes.

An important feature of the atrium is the arrangement of sunscoops, one external and one internal, that reflect daylight into the building and to light the internal atrium space.

Towards the top of the building are the senior executive suites, together with the Chairman's apartment. Above levle 43 the building structure asserts to form the 'top of the building', a group of loors containing and executive areas surmounted by a helicopter landing pad.

Sub-
Structure

The planning of the basements reflects that of the superstructure, with vertical services distribution, lifts and stairs concentrated on the east and west sides (Fig. 10). Generally, four levels deep, the basements reduce to a single level on the west side because of the retention of the annexe building during the first phase of construction.

2.0 Erection Sequence

The erection of the building structure was basically zonal, each bank of floors and its suspension trusses being essentially structurally complete before handover to following trades. Within a zone the central area floor steelwork was erected closely behind the masts, rather than waiting until the suspension trusses for that zone were complete. This gave significant programme advantages.

Until the suspension trusses were complete, therefore, the hangers had to act in compression and be temporarily supported from below. For this a trestle was place under each pair of hangers, the tresstles being supported by the work platform for the erection of the lowest zone and by the suspension structure below for higher ones. Following completion of welding in the suspension elements, the trestles below were removed during the 'jackdown' operation, whereby the structure was initially jacked-up off the trestles to remove the supporting packs and then let down until it hung freely.

Steelwork erection above continued within prescribed limits before completion of suspension trusses and jackdown of the zone below. Concreting of the floors was carried out after jackdown. Completion of the suspension trusses also allowed the erection of the outer hangers, floor elements,and stairs within a zone. Module erection was completed in a narrow time window and marked the essential structural completion of a building zone. The stages are illustrated in Fig. 49.

The lowest, most slender hanger sections were stiffened to increase their compressive capacity to support the 'jackdown' loadings. Jacking forces and displacements were closely monitored during the operation to give warning of potential overloading of the hangers. The load/displacement records gave an indication of the stiffness agreed closely with predictions.

3.0 Cladding and curtain walling

Visual expression of the building's structure is a fundamental

architectural consideration. Early studies incuded extensive investigations of various methods of providing corrosion and fire protection to fully exposed external steel members which would then remain unclad. Possible corrosion protection solutions included the use of weathering steels and corrosion-resistant steels. The use of molecularly bonded stainless-clad steel was also investigated in detail, but there were significant technical difficulties in the material development. Fire protection studies for these schemes included intumescent coatings, flame shielding, and water filling. It was concluded at that time that such tital exposure of a large structure would be impractical given short timescales available, and the solution was adopted whereby the structure would be fully expressed, but clad.

The design of the various cladding and curtain walling systems is complex. In addition to the stringent visual criteria, the varying plan geometry of the building and the exo-skeletal structure create a great number of different details and many penetrations through the curtain walling.

Having established the criteria for appearance, form, and structural performance, the design of the curtain walling systems was developed throught the evaluation of models, mock-ups, and prototypes, with the design team working in close cooperation with the contractor in Hong Kong, London, and at the fabrication works in the USA.

The typical curtain wall consists of storey-height 12mm-thick fully tempered clear glass between vertical aluminium mullions.

the mullions are suspended from each floor via sand-cast aluminium brackets, with vertical movement joint at the junction to the mullion below. The typical mullion is stiffened by the addition of a Vierendeel truss to span the double-height floor spaces.

The cladding to structural members comprises 6mm-thick aluminium sheets with extruded aluminium edge sections and stiffeners which are plug welded to the sheet. The cladding panels are fitted to the structure by bolting to prefixed stainless steel channel sections, which are fixed onto stainless steel studs welded onto the steelwork. A layer of cement-impregnated tape is located between the channel and the carbon steel structure and a silicone seal applied around the heads of the connecting studs to prevent bimetallic corrosion.

The service modules and risers are clad in aluminium, laminated honeycomb panels. All of the external and visible aluminiuum surfaces received a sprayed, pigmented (onyx grey), fluoropolymer finish, applied at the works before shipment, wwhich gives the building its final appearance.

The stair shafts are fully glazed using structural silicone sealant to secure the glass to the internal mullions. all such structural silicone was applied in controlled environmental conditions. Particular attention was paid to surface preparation and cleaning, and regular sample adhesion testing was carried out.

4.0 Movement effects: curtain walling

In concept the cladding elements are suspended panels fixed to the floor structure with adjustment in the fixings to allow the panels to be set level at the time of installation. Allowances for the relative vertical movements of the floors at the head and base of each panel are provided in the base fixing detail, with the movement allowance varying according to the panel's location within the building. Since cladding installation followed closely behind steelwork erection in each zone, it was particularly important to assess the structure movements which would occur between installation of the cladding and completion of construction.

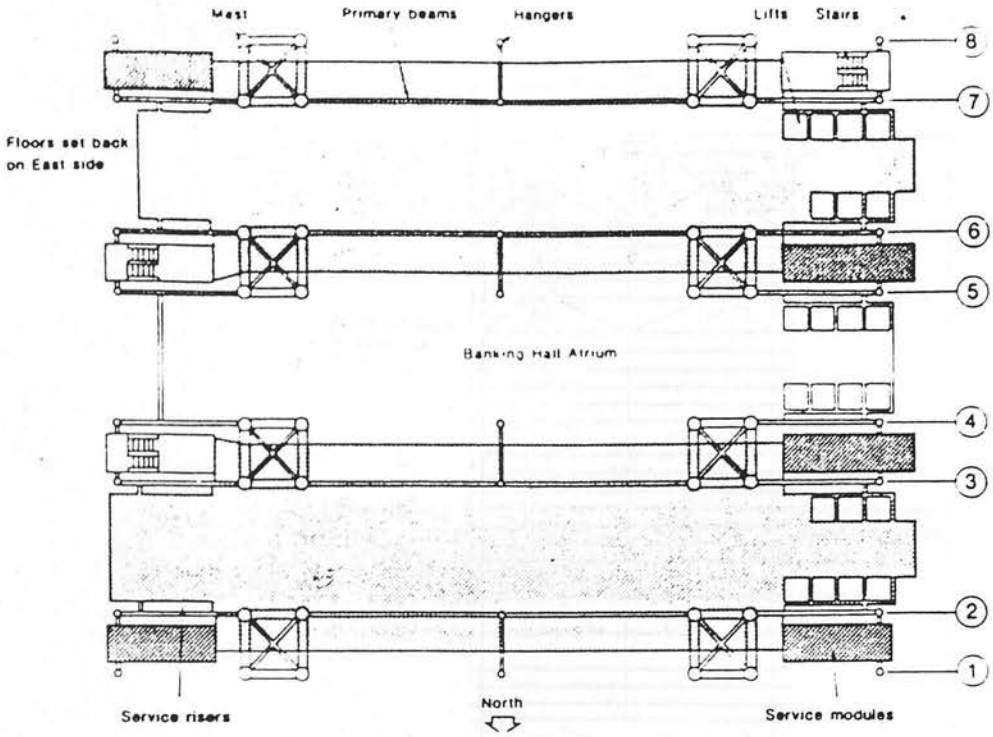
Movement allowances had been incorporated into the panel design at an early stage, and these could now be checked from the current analyses which used detailed information on the weights of fabricated and manufactured items, together with the final erection programmes.

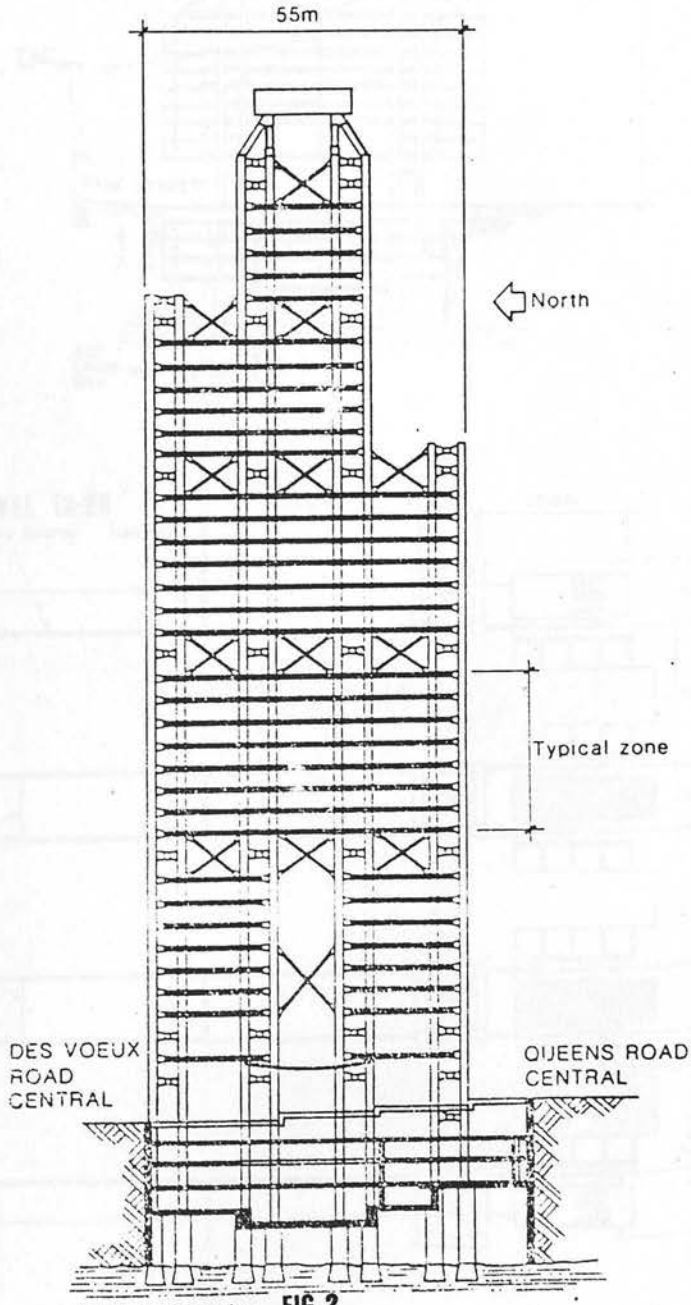
The analyses provided the minimum movement capacities that would be required on erection, determined from:

- floor deflections and main frame support movements occuring after installation of the cladding because of progressive construction and fitout of the building.

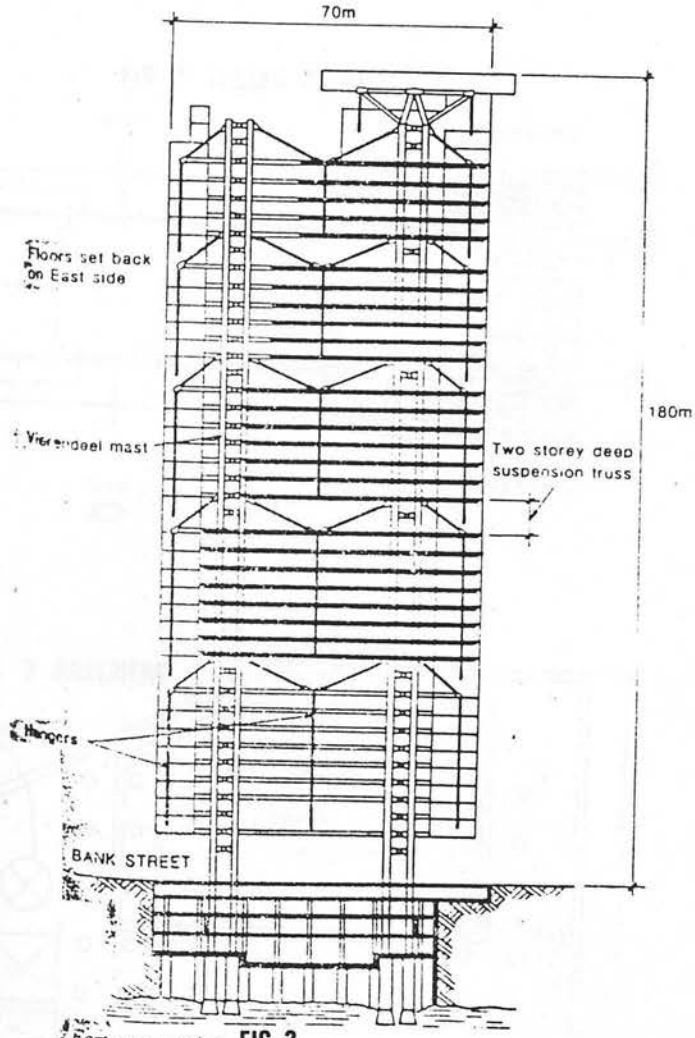
- floor deflections due to the superiimposed live loading.

FIG 1 LEVELS 3-11





North-south section **FIG 2**



East-west section **FIG 3**

FIG 4 LEVEL 13-28

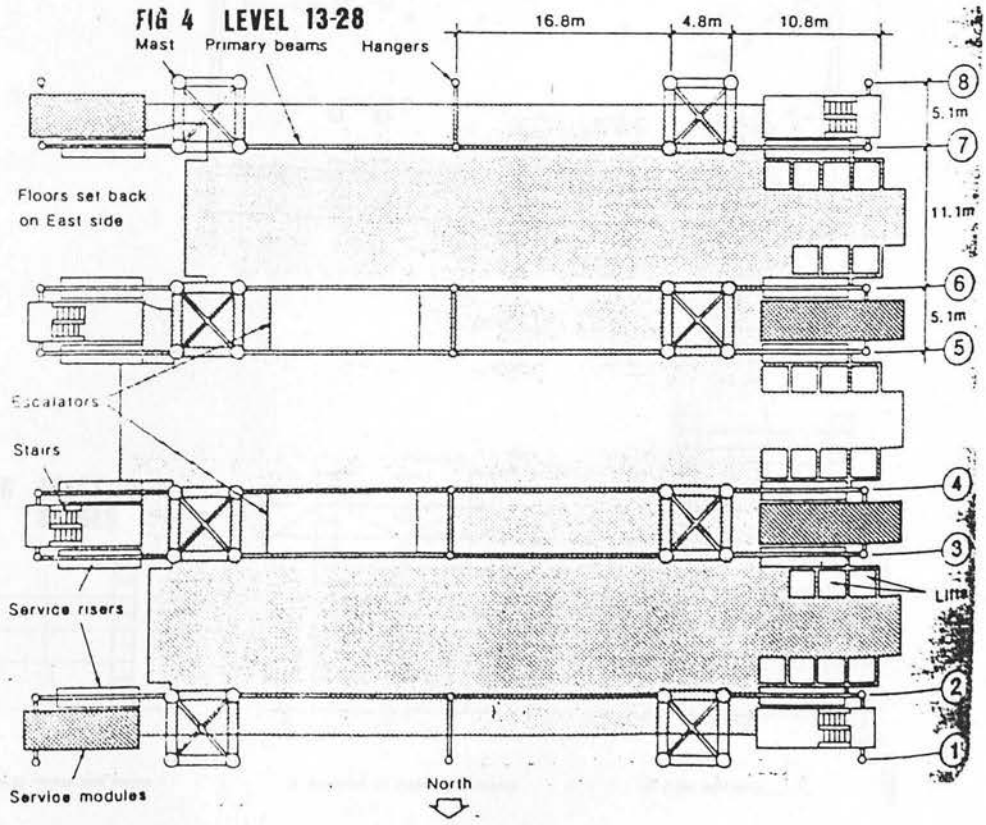


FIG 5 LEVELS 37-41

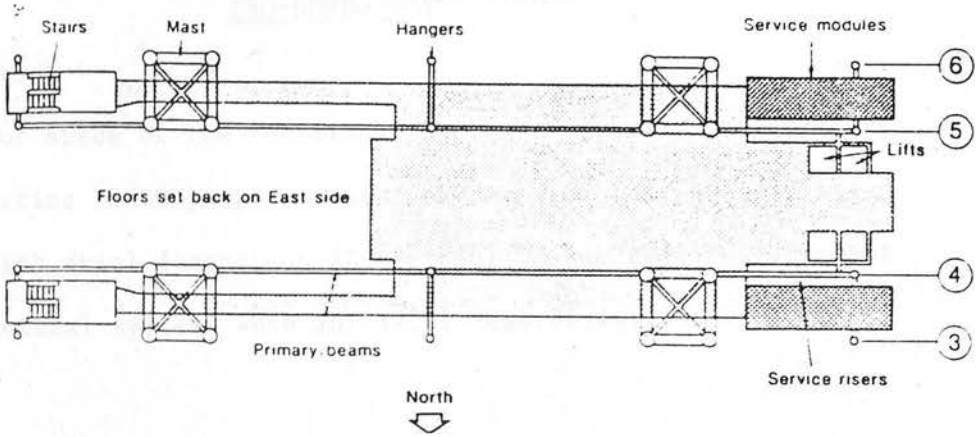


FIG 7 BASEMENT

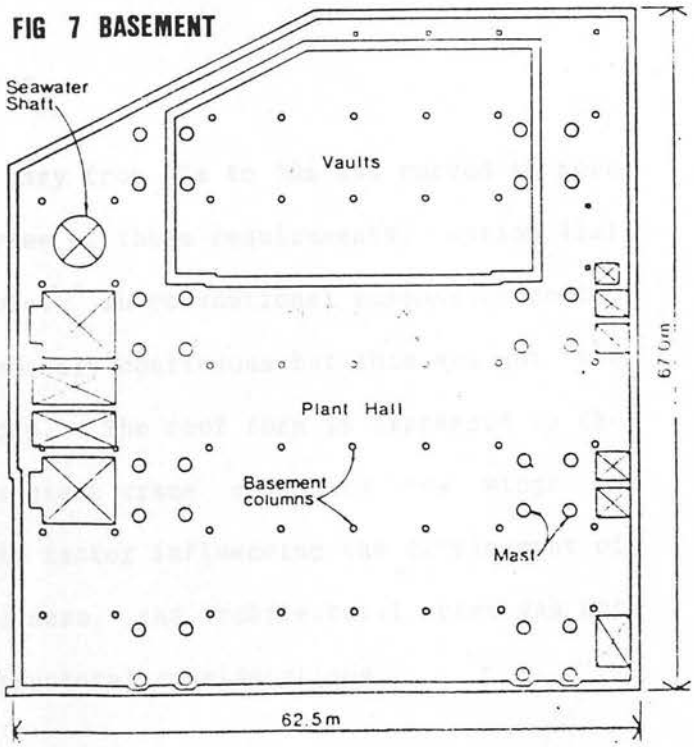
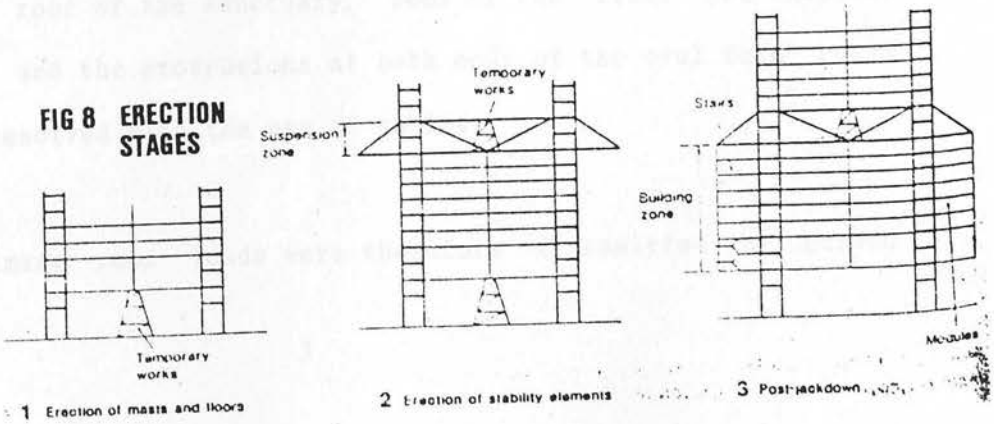


FIG 8 ERECTION STAGES



1 Erection of masts and floors

2 Erection of stability elements

3 Post-jackdown

CASE STUDY No. 41

SHO-HONDO ROOF

1.0 Introduction

The major space of the religious complex comprises a sanctuary accommodating 6000 people. Studies revealed that an oval plan form with axial dimensions 82.5m and 110m was required and that two structural systems were initially considered:

(a) Suspension system

(b) A space frame truss.

2.0 Architectural Considerations

The ceiling height was to vary from 60m to 30m and curved in more than one direction. In view of these requirements, option (1a) was considered more suitable. In conventional suspension roofs, the curved planes were smoothly continuous but this was not the case with the expressed form. The roof form is expressed in the metaphorical image of a giant crane spreading its wings in flight. This was the main factor influencing the development of the sturcture and in this case, the architectural space was not initially determined by structural considerations.

3.0 Structural Considerations

In the roof of the sanctuary, some of the lines are acutely angular and the protrusions at both ends of the oval form could not be resolved with the use of cables.

The primary roof loads were therefore transmitted by curved

beams, in tension and in bending to a system of peripheral rim beams. A tension ring is located in the roof centre and the curved beams form a compression ring. The two rings are connected by radial beams supporting the roof covering and the entire system may be understood as a variation of a spoked wheel structure. The peripheral rimbeams are bent at nearly right angles on the short axis of the oval and are not able to withstand the thrust of the roof. A system of pyramidal trusses was conceived to resist the thrust of the radiating curved beams from the roof. These pyramidal trusses are spaced on the periphery of the building rising full height from ground level to eaves, meeting with the peripheral rim beams, providing the architectural character to the interior space and to the building form.

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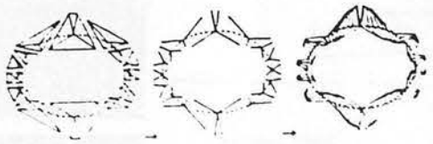
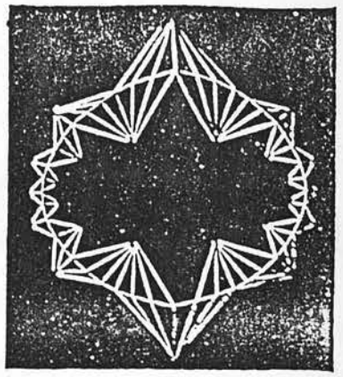
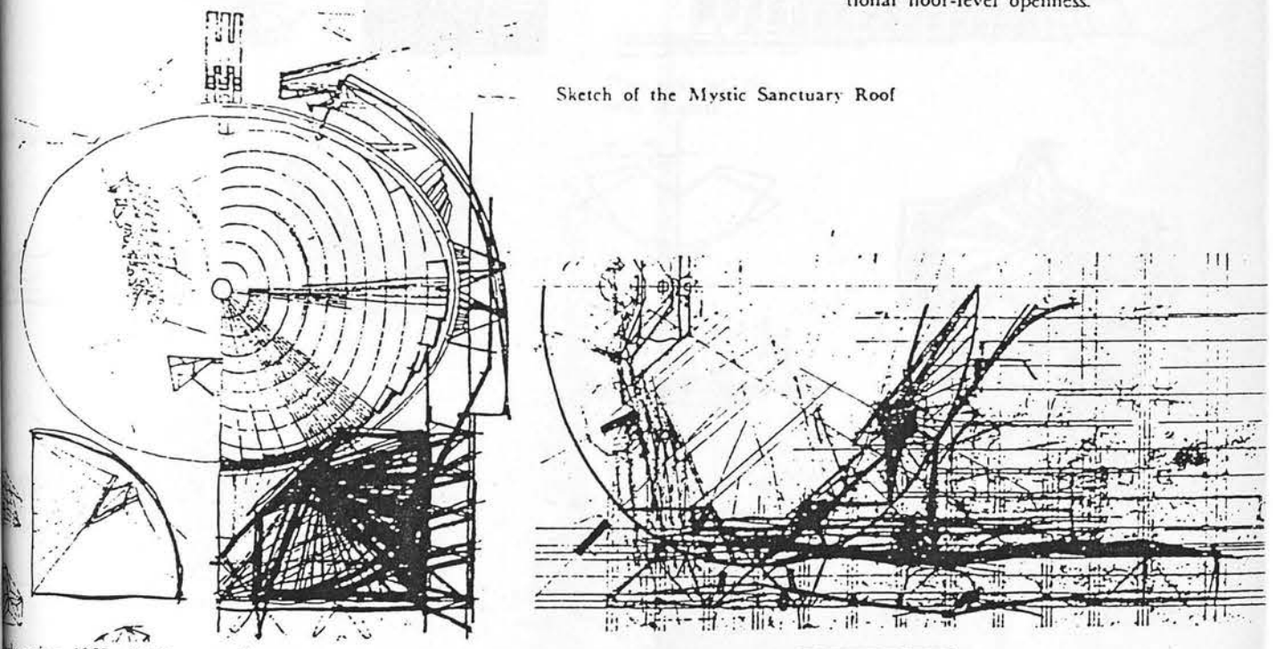
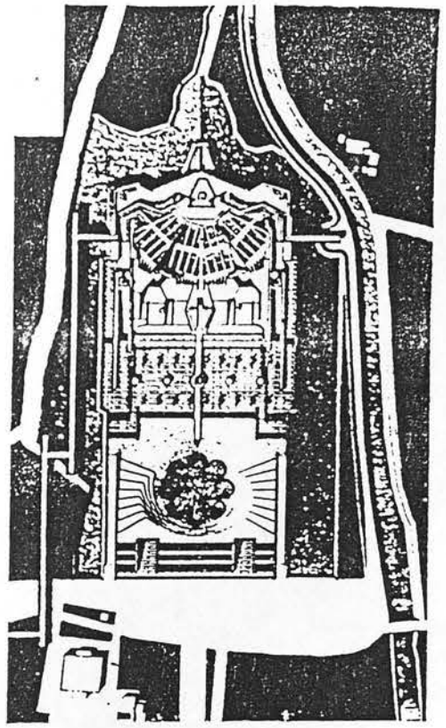




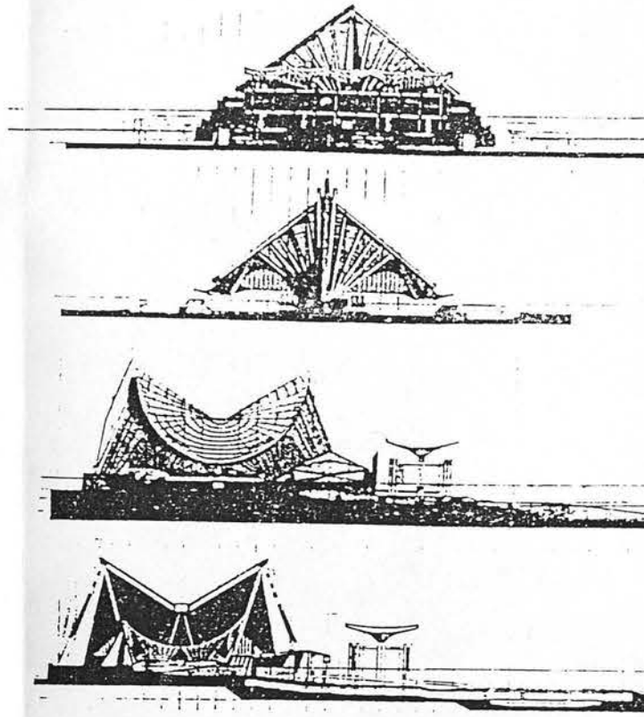
ecause of the inward-directed nature of
e roof and the need to produce maxi-
um resistance against horizontal stresses caused by seismic tremors, our research on the dynamics of the lower structure began with the idea of triangular pyramidal units. The negative spread of these fan-shaped supports satisfied the requirements of structural logicality and of functional floor-level openness.

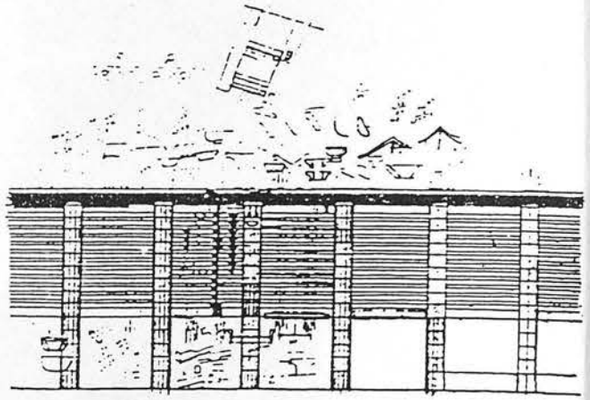


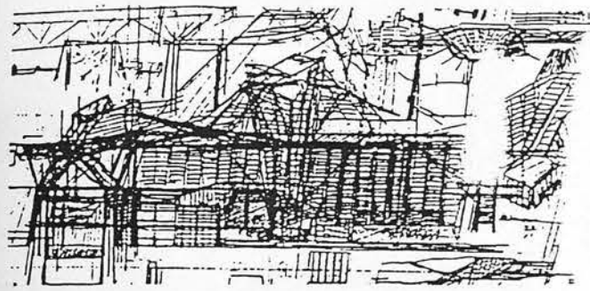
Sketch of the Mystic Sanctuary Roof

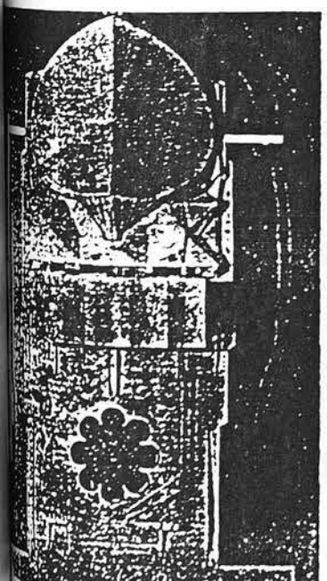


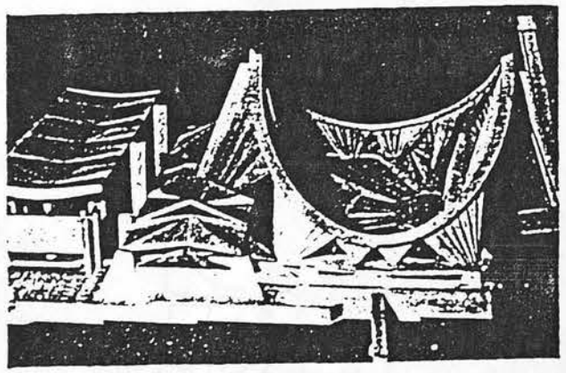
Basic plan of the third proposal.

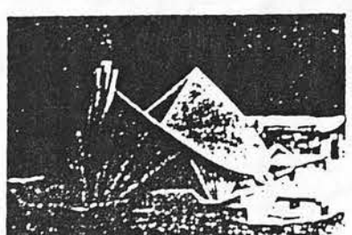


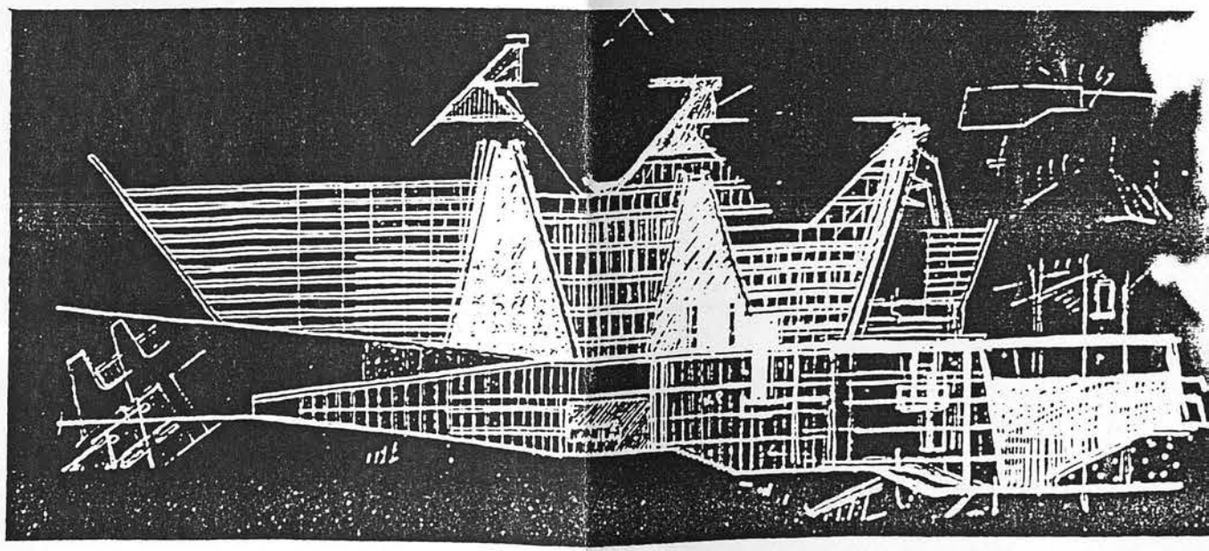








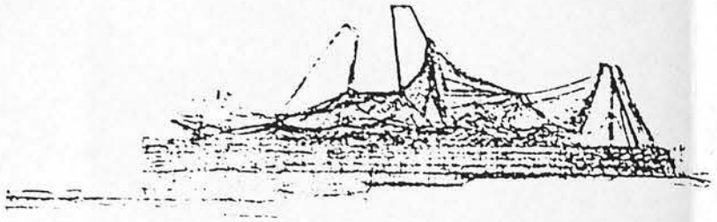


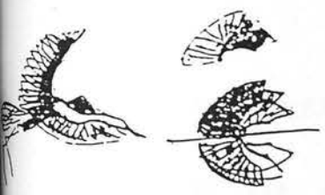


ly spring, 1967; third proposal

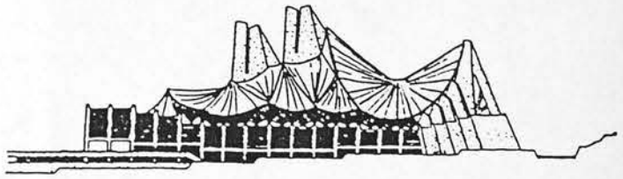
mn, 1965

his point, we drew sketches of the roof symbol of a crane with outstretched wings. This image was selected because it symbolizes the universe and the hopes for the world held by religious people.





Spring, 1966



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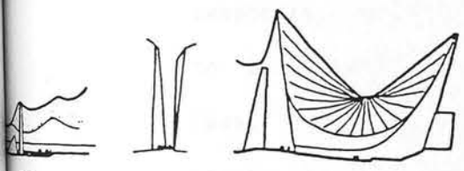
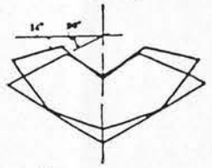
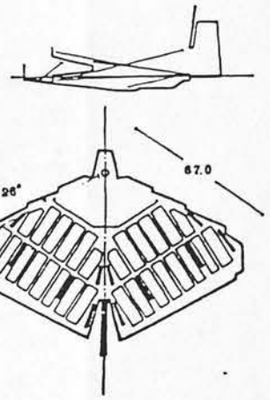
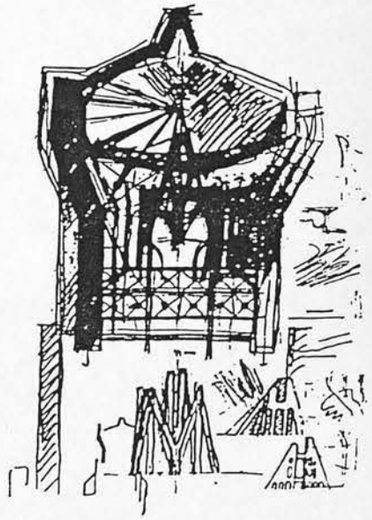


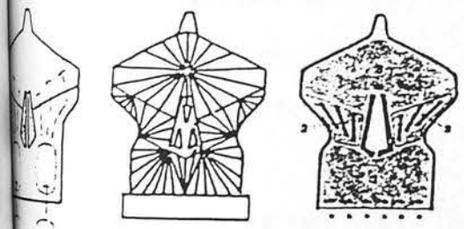
1 2 3

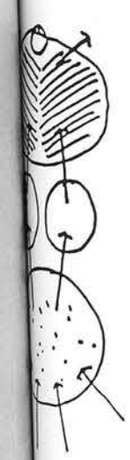
Placement of the rows of seats

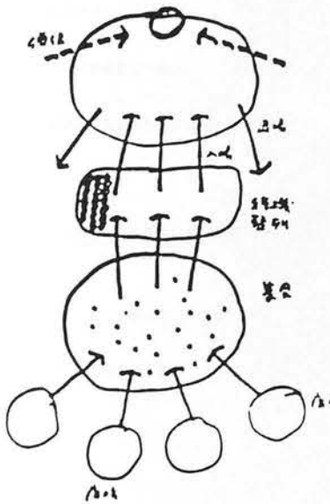


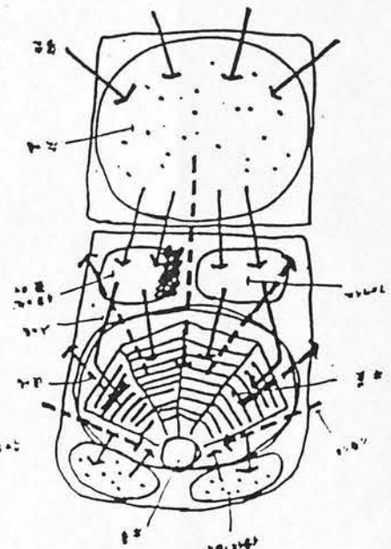


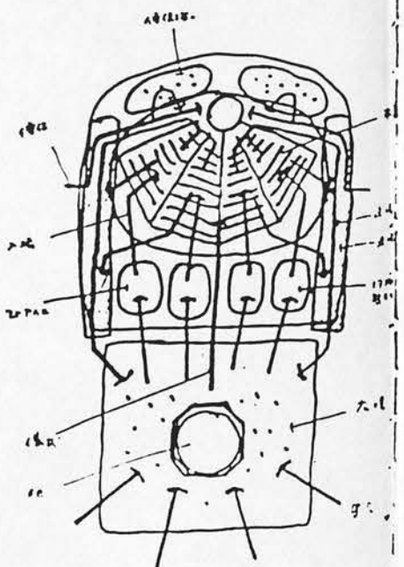




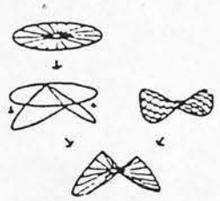


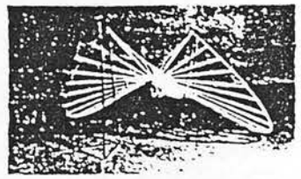


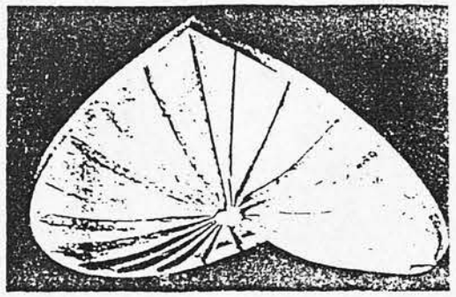




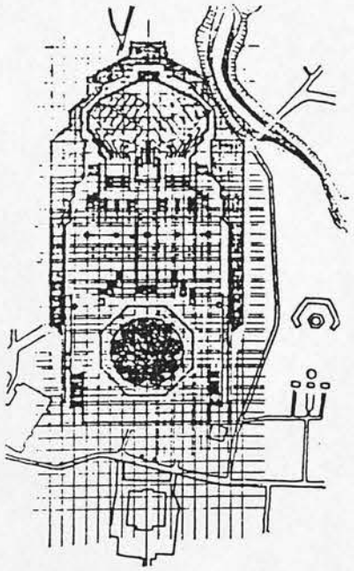
Roof of the Mystic Sanctuary

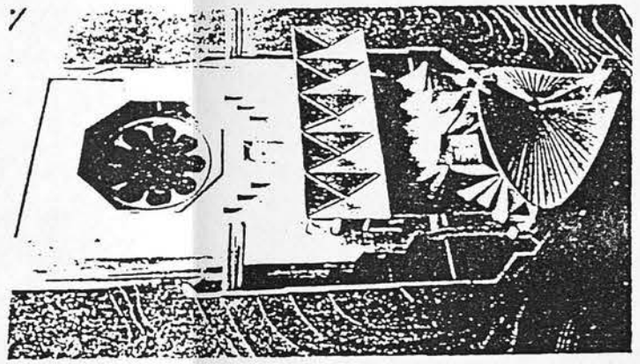


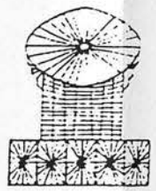


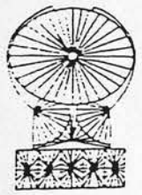


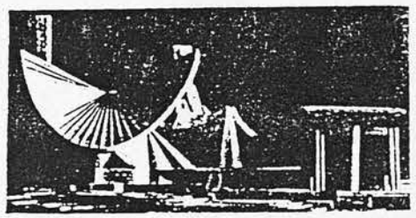
First proposal



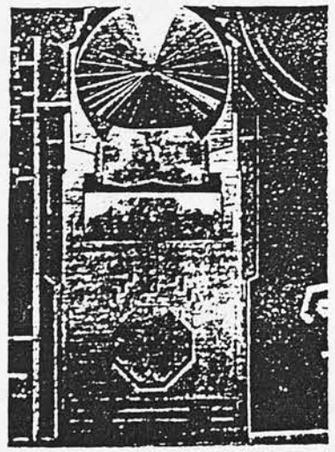


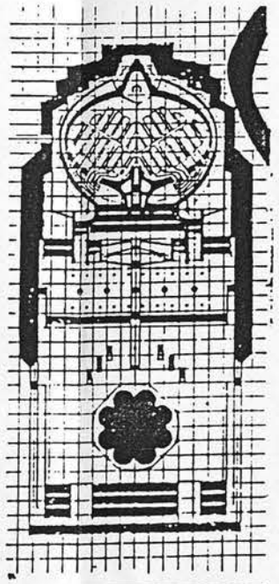




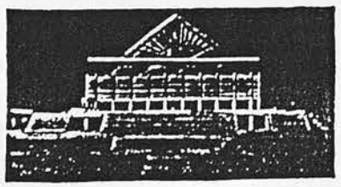


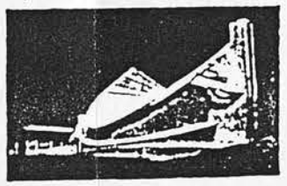
Early summer, 1966; second proposal

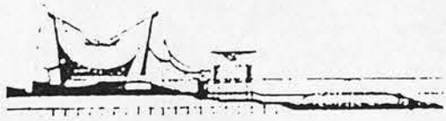




Symbolic aspects of the form of the o plan roof (long axis 110m, short 82.5m) include the image of a bird mo its great wings, wavelike motion, and versal transmigration of souls. Dy cally, the following aspects of the ture demanded meticulous care: tran sion of forces, vibration caus quakes and winds, and th always a major issue in sus







CASE STUDY No. 42

LES TOURS DE LA LIBERTE

Architects: Jean-Marie Hennin & Nicolas Normier

Extracts from a report by: Bernard Vaudeville, OAP
Brian Foster, OAP

1.0 Introduction

It was decided a long time ago that the celebrations for the Bicentenary of the French Revolution would be marked by a temporary, but imposing, monument in Paris. Originally, it was to be a World Exhibition, on which Arups began work in 1983. Then, owing to repeated changes of programme, soul-searching about the significance of the anniversary, and shifts in the balance of power in the French Parliament, the Government were in the position of having to choose a project as late as October 1988, to be completed before April 1989.

The scheme that was finally selected had the advantage of having been drawn up quite carefully by the architects Hennin and Normier, as well as being supported by a steelwork contractor, Viry (who, incidentally, built two other Arup projects - a Commercial Centre near Nantes by Rogers and the Nuage at Tete Defense, Paris).

2.0 Description of form and architectural concept

It consists of two 35 m high towers, situated in the Jardin des Tuileries in front of the Louvre, within a fair commemorating

events during the Revolution.

Each tower has a central pylon, 4.5 m x 4.5 m square on plan, carrying at a height of 12.5 m a two-storey cantilevers box 18.6m x 15.6 m on plan. The two boxes house a radio studio, information stand, party reception rooms and, at roof level, observation decks. In essence, the construction is of welded hollow sections with solid round-bar bracing.

The towers also serve as a framework to which layers of wings, fins and canopies - each covered by semi-transparent materials - can be attached and suspended in mid-air. Indeed, above all the towers are sculptural landmarks, each crowned by two imposing wings which can be seen from a long way down the River Seine. The project explores space and air by means of an open, even exploded form, with the characteristic shapes of early 20th century aeroplanes serving explicitly as formal references for Hennin and Normier.

3.0 Influences on development of structure

Ove Arup & Partners, through the office of RFR in Paris, did the technical studies and the conception of the main details for the steel structure and the wings, in close collaboration with Viry.

The main technical difficulties that had to be overcome are summarized as follows:

3.1 The tight schedule

This was the overriding challenge and a strategy had to be established so that Viry could start manufacturing while the engineers were still designing. To this end, the structure was split into three sections: the central pylon, the glass box, and the hung elements (wings, fins, and canopies).

Outline schedule

End of November 1988
Beginning of the technical studies

15 December 1988
Sections and forces in the central pylon established

24 December 1988
Structure of the'box resolved

15 January 1989
Changes in the structure of the 'box', at Viry's request, owing to manufacturing difficulties

15 March 1989
Detailed calculation of wings and canopies completed

3.2 Computer models

Given the close interaction of these three sections, various overlapping computer models were created by the engineer to determine the forces in the pylon.

Where the pylon penetrated the box, the bracings were omitted to allow free circulation. Here, the structure of the box itself had to provide the bending and torsional rigidity to the tower. At this stage the rigidity of the box was set very high, by means of simplified bracings, in order to lower the torsional dynamic response of the pylon.

On this model the engineer defined the structure of the box according to the architects' requirements (with as few visible cross-bracings as possible) and in order to achieve the previously set torsional rigidity of the whole tower. The concrete floors were used as rigid diaphragms between the pylon and braced external facades of the box.

3.3 Dynamic response

The bending and torsional dynamic behaviour was the main weakness of the principal structure, because of the absence of any cross-bracing where the pylon passed through the box.

4.0 Structural Development

The wings on top of each tower are large enough and high enough to be seen from many points around the city. Each wing is a segment of a circle 31m long, 10 m deep, and inclined at 15 degrees from the horizontal. Each is a thin plane covered with a white PVC-coated polyester membrane perforated to about 30% of its area. At close range this gives a degeee of transparency that reveals elements of structure beyond.

The architects had determined the outline dimensions of the wings and Arups role was to add substance to them in developing a structure which was constructable yet did not lost the 'esprit' of the original concept.

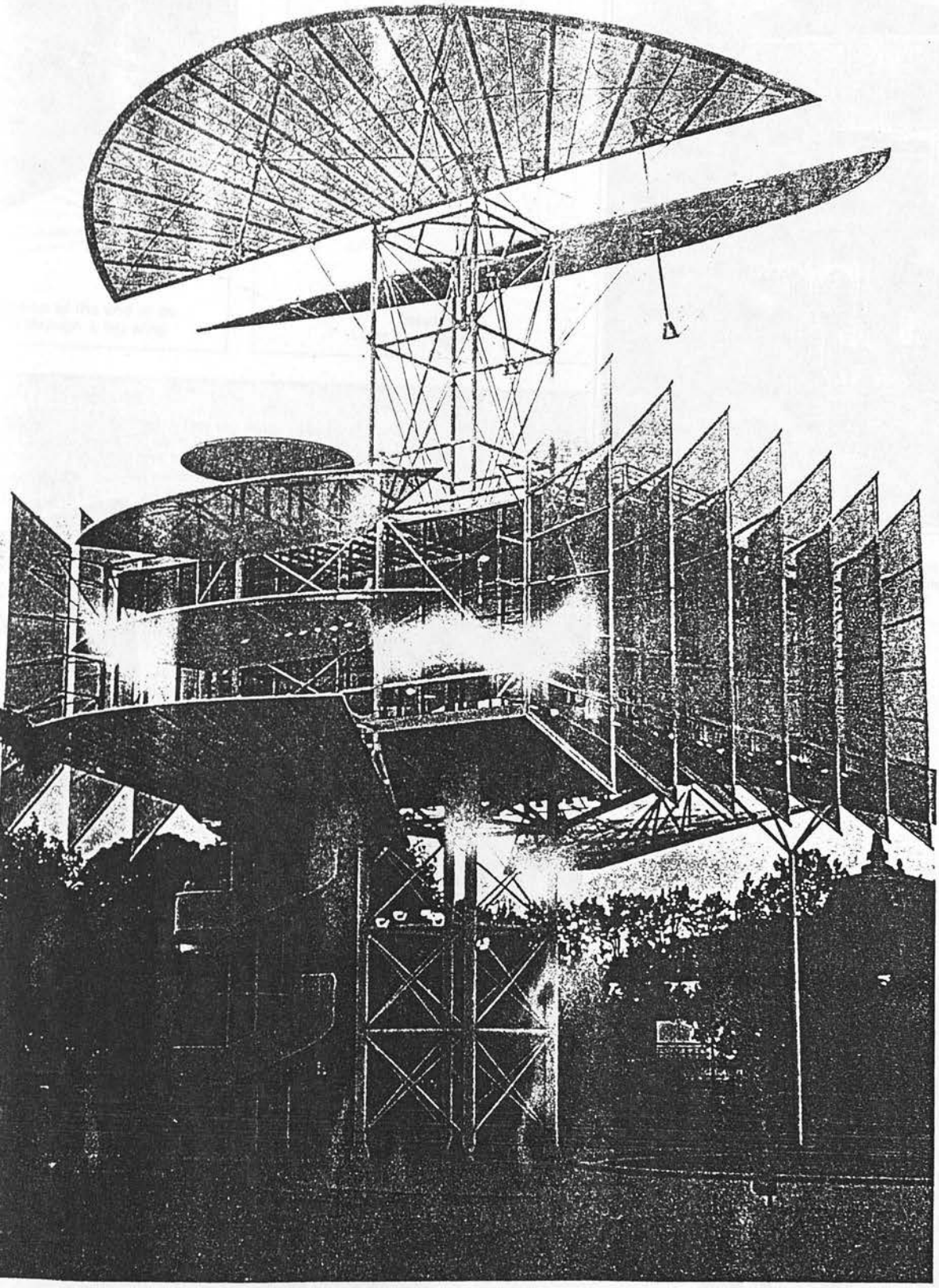
The wings had to be seen as thin planes appearing to have minimal contact with the pylon. It was also clear that in the short time

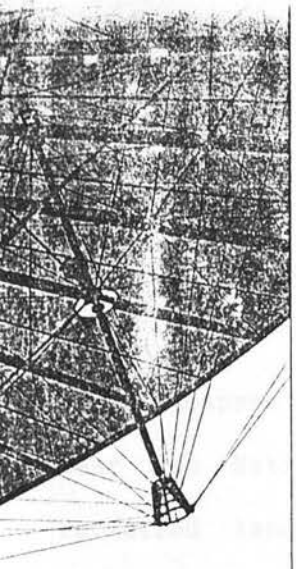
available for construction, each wing would have to be assembled complete at ground level and lifted by crane onto the tower. Thus the structure of each wing would have to be stable within itself before being attached to the tower.

The engineers therefore introduced the use of flying axles that passed through the surface of each wing and from the end of each axle a fan of 16 mm diameter tie-rods running out to the 168 mm diameter circular edge beams was arranged. This system provided the out-of-plane bending stiffness needed in each wing. In-plane stiffness was provided by radially aligned strut/beams 139 mm diameter CHS with 12 mm diameter rod cross-bracings. To reduce the bending imposed on these beams there were fans of tie-bars from the ends of each axle that picked up the mid-point of these beams, forming a tension spine in a curved plane. This spine enhanced the wings' out-of-plane stiffness and reduced the diameter of the beams. Modest prestress was introduced into the tie-bars sufficient to maintain the straightness of each bar in its erected position. The straight edges of each wing were hinged directly onto the corners of the towers. Each wing was then held against global rotation by further tie-bars from above and below.

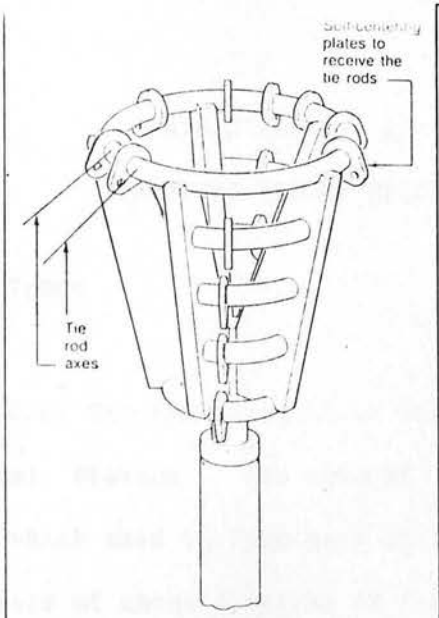
The membrane was manufactured in a single piece from strips welded together following the lines of the radial beams. A prestress of approximately 300 kg/m was put into the membrane by pulling its exterior boundary onto the perimeter beams of the wing. The extension in the material to get this force was allowed for in the cutting patterns that the engineers supplied to the contract. Having clamped the fabric to the edge beams it

was then fastened continuously along each radial beam using a rope lacing detail. This was done to limit the deflection of the membrane surface under combinations of wind and snow load.

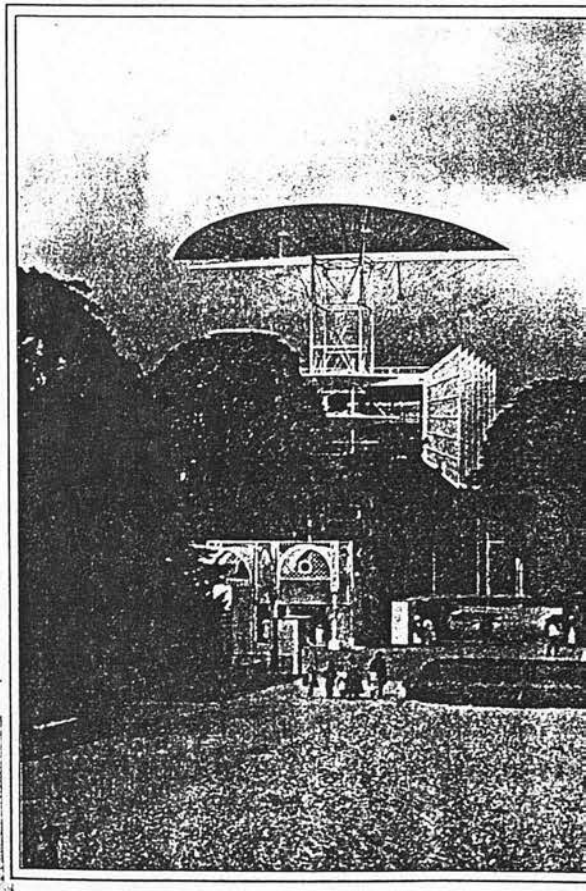




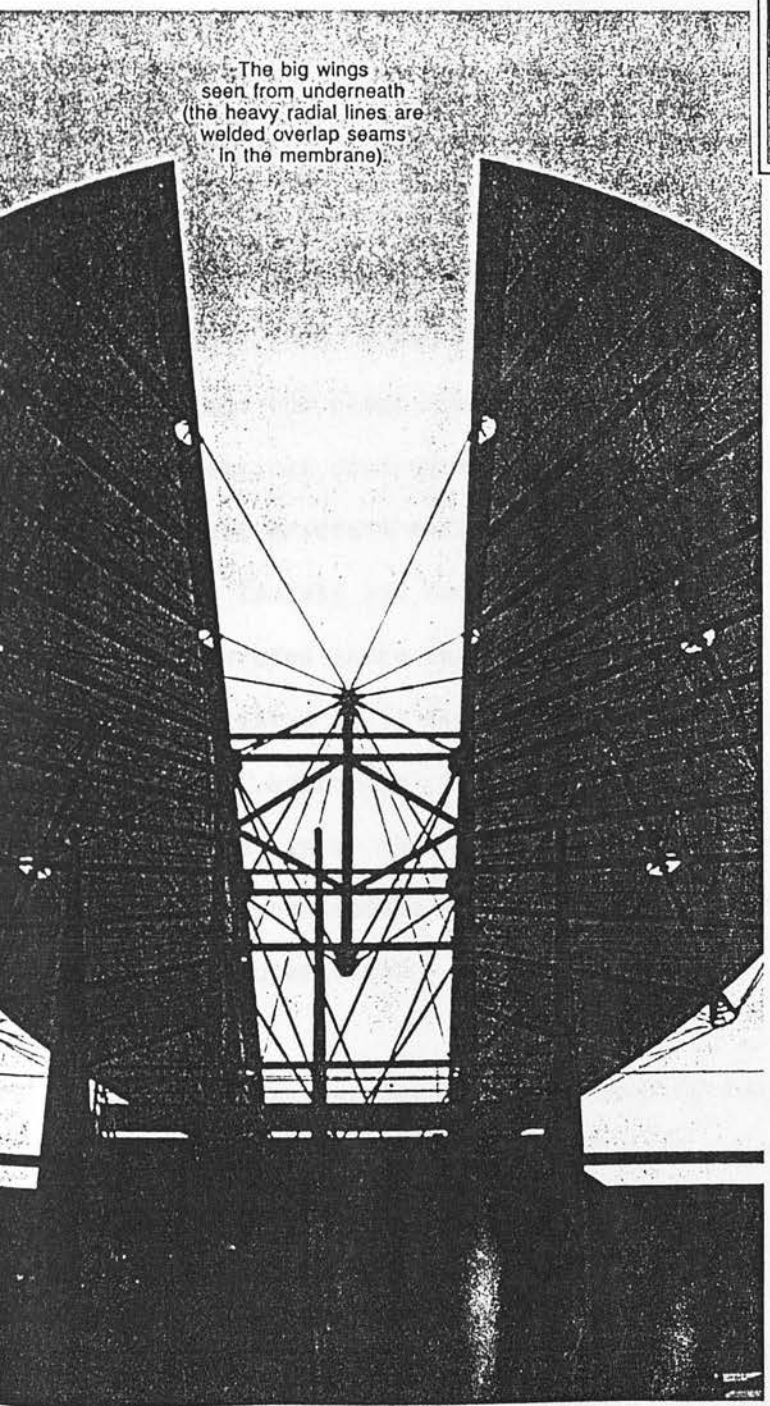
lose-up of the end of an
xle through a big wing.

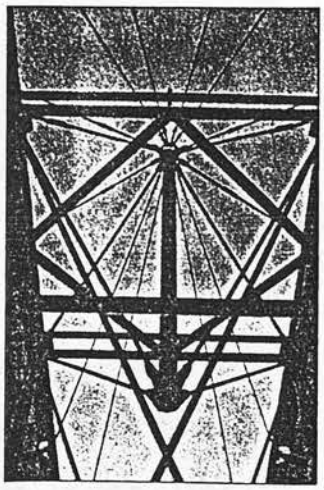


Detail of
end of flying axle.



. View from the
entrance ramp of one
tower across the Tuileries
to the second tower.





Above: Flying strut
at the top of the pylon.

CASE STUDY No. 43

SINGAPORE INDOOR STADIUM

Architect: Kenzo Tange

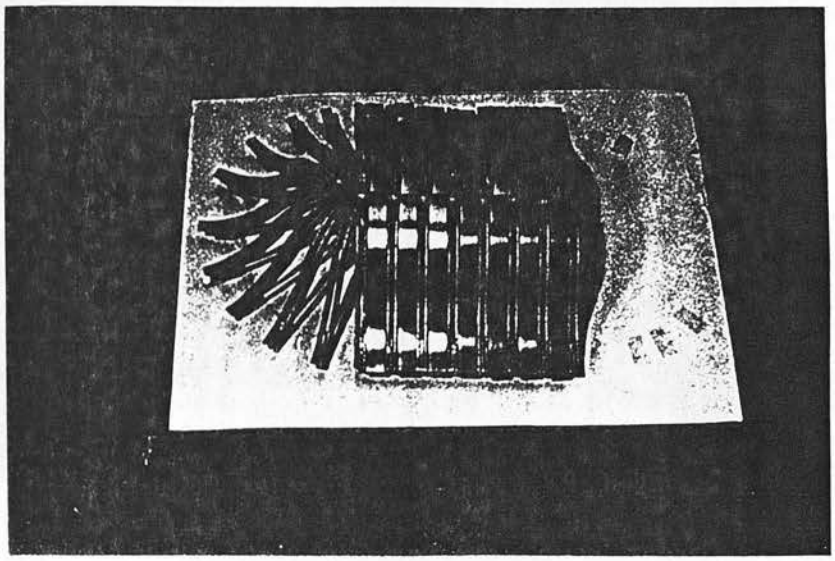
The Singapore Indoor Stadium is sited in the Kallang Sports Complex, near the National Stadium. The site of the Indoor Stadium is on reclaimed land which used to form part of the old Kallang Airport. The ground consists of about 4 metres of fill material on top of 12 metres of soft marine clay. More than one thousand 30-metre long steel piles had to be driven to carry the weight of the stadium.

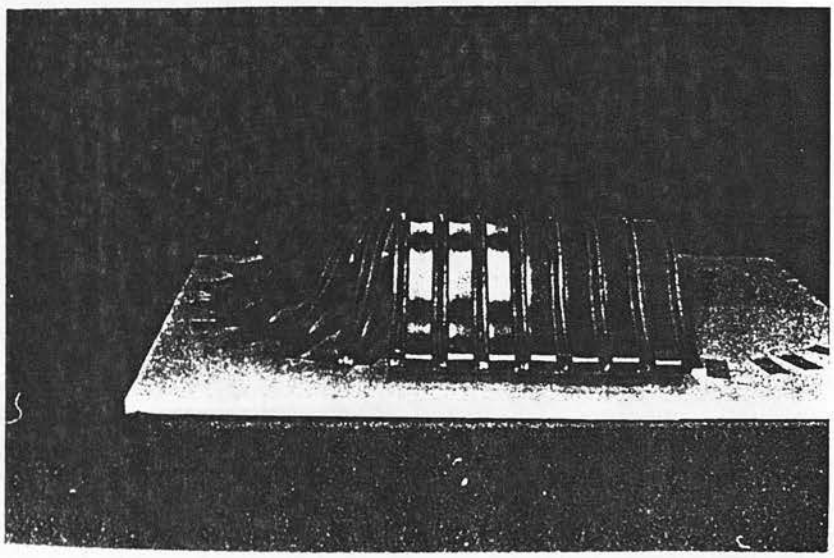
The principal feature of the design is the diamond-shaped plan and convex profile of the roof. The roof being the most complex part of the stadium structure accounts for a major portion of the building cost. The 1500 tonne roof is supported by tubular steel perimeter columns and the clear span of the building is more than 100 metres. The geometrical configuration of the roof required some innovation in conventional erection methods. As with the Barcelona Stadium designed by Arata Isozaki and currently under construction, the Pantadome system was used where the roof structure is assembled at ground level and then lifted up to its final position with a system of carefully positioned hydraulic jacks. A number of special hinges are incorporated into the roof structure and its supporting perimeter columns to accommodate their large displacement and rotations during the lifting operation.

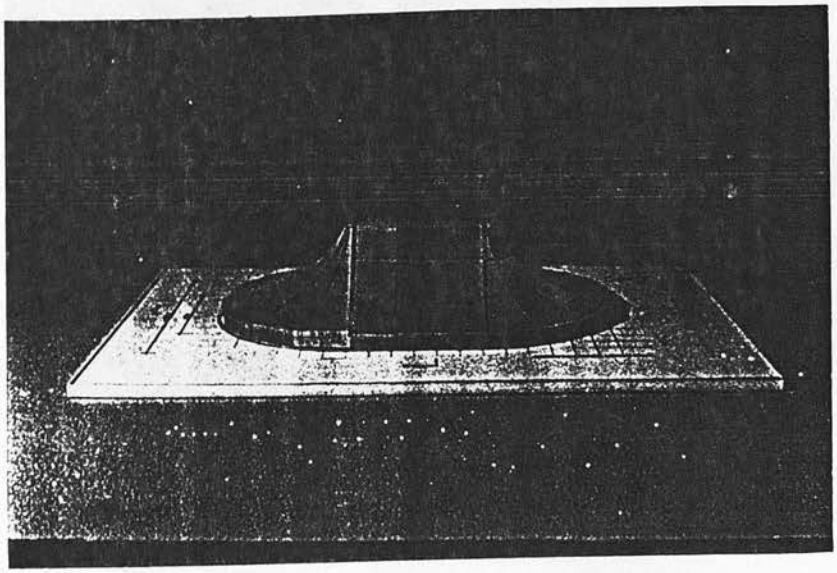
All lifting operations were monitored by computer to ensure that the roof structure did not become unstable or overstressed at any time. When the roof had reached its final position, the hinges in the roof

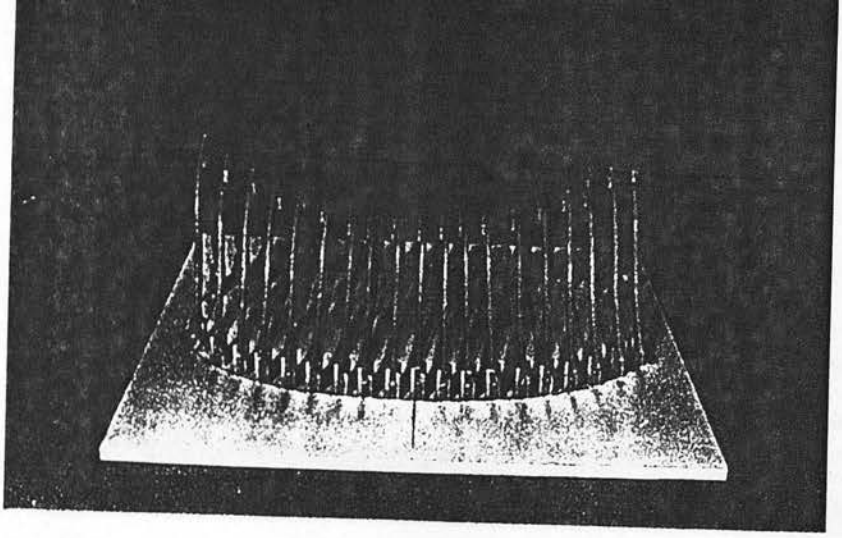
were locked by the insertion of additional space-frame members.

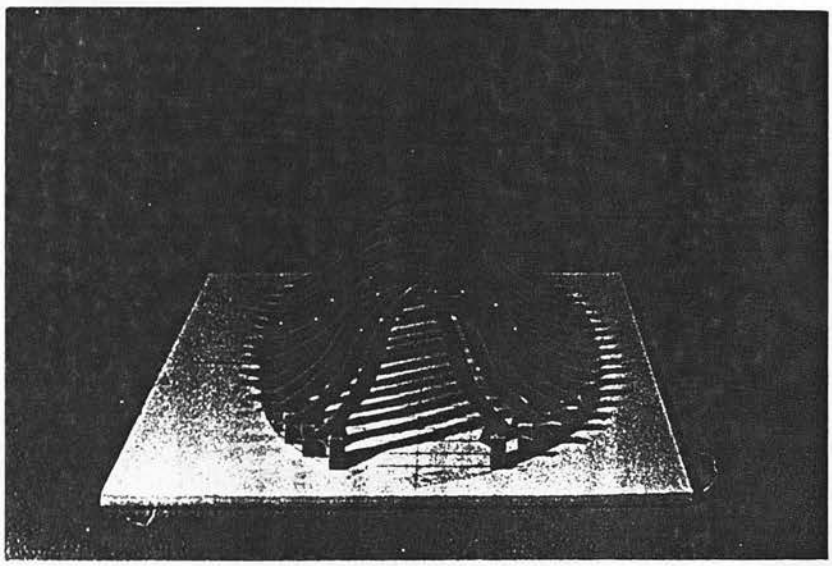
The Pantadome system allows the advantage of the installation of roof cladding elements, mechanical and electrical networks at ground level, prior to lifting. The ridges which separate the four sections of the roof admit natural light. The roof is clad with a continuous system of stainless steel sheets. The ceiling is finished with perforated aluminium panels and follows the profile of the roof.

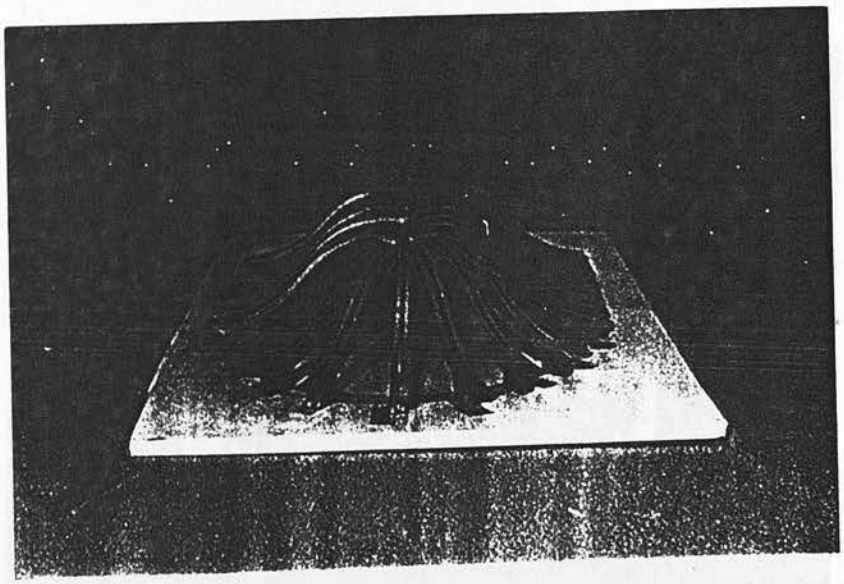


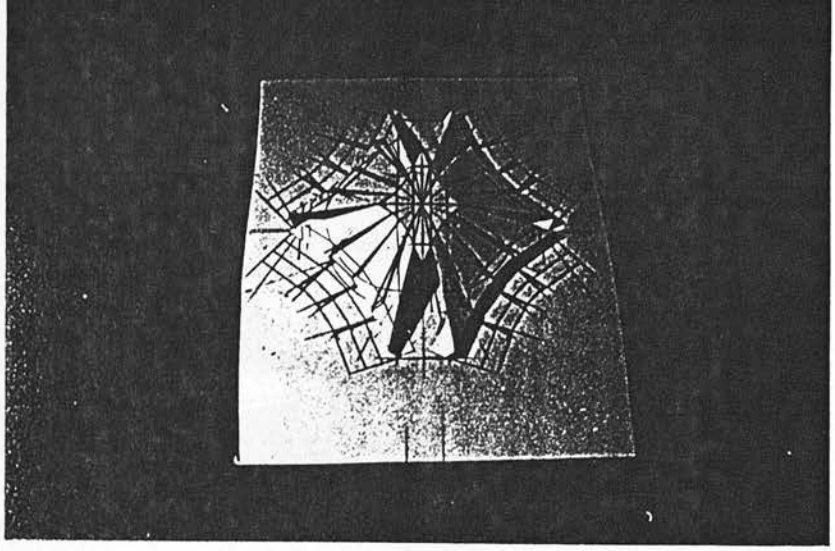


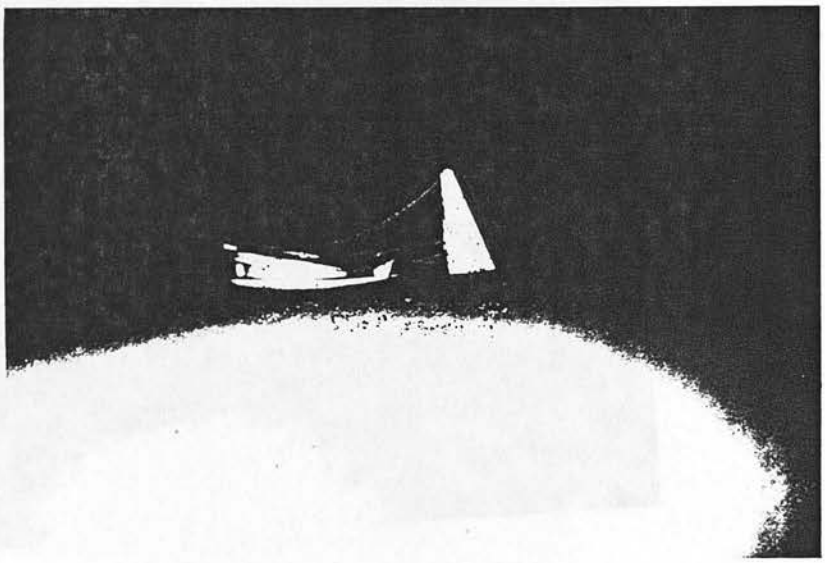


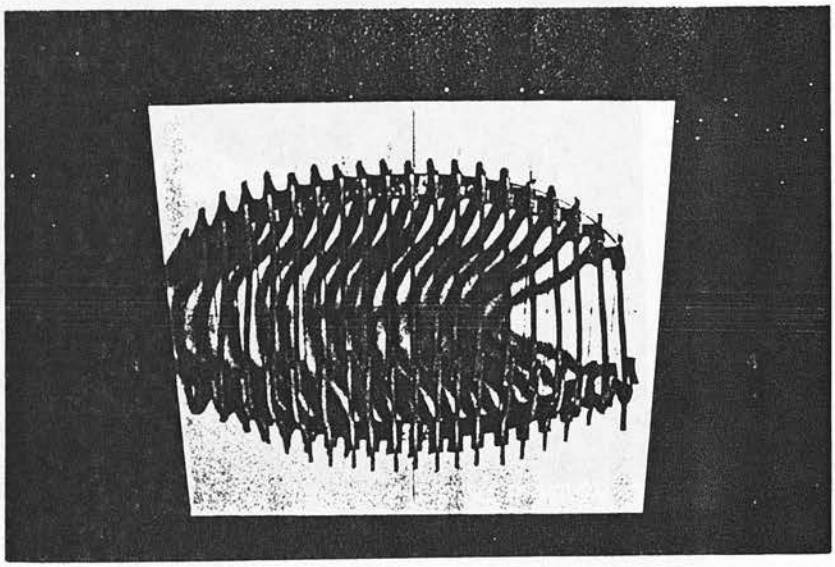


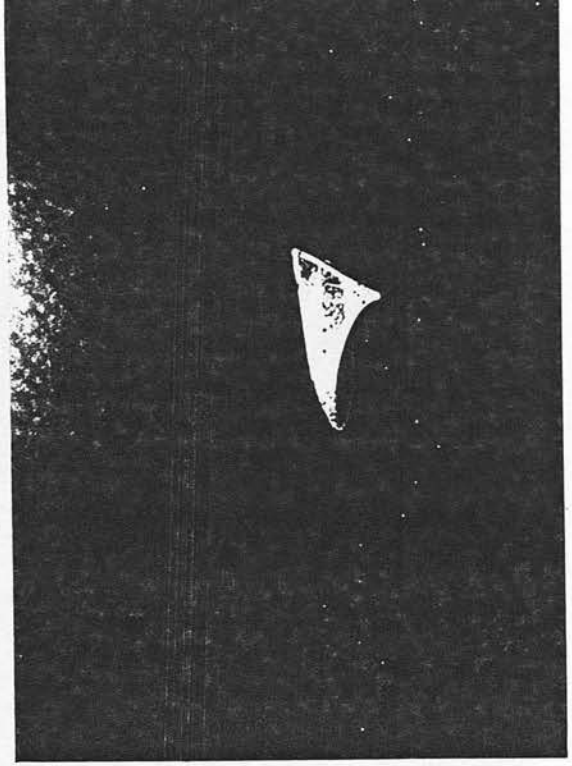


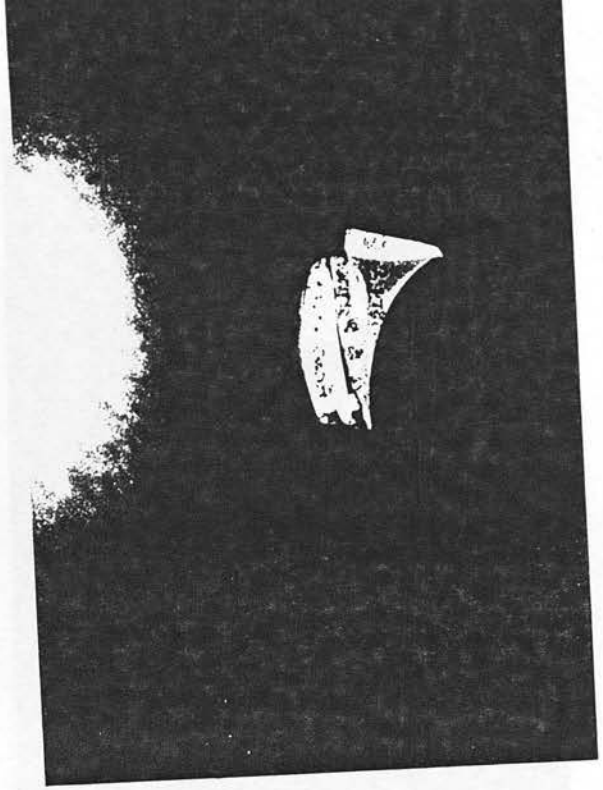


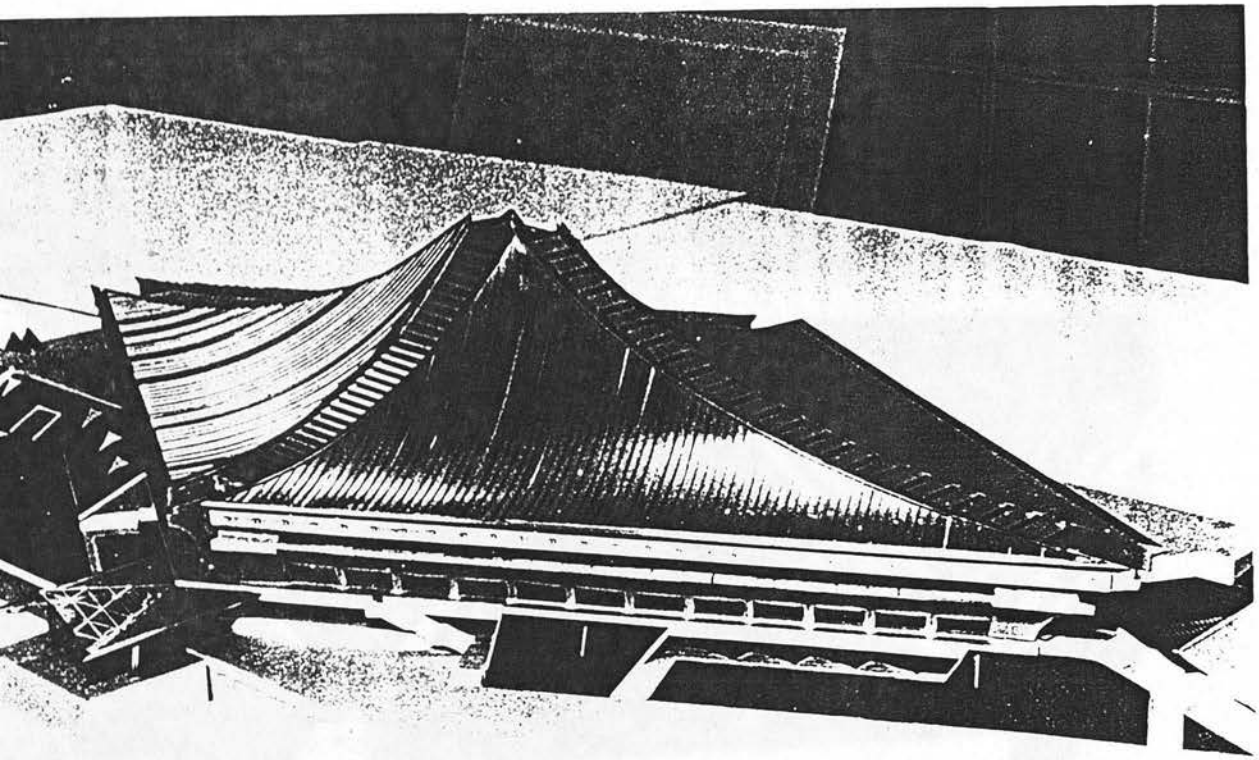


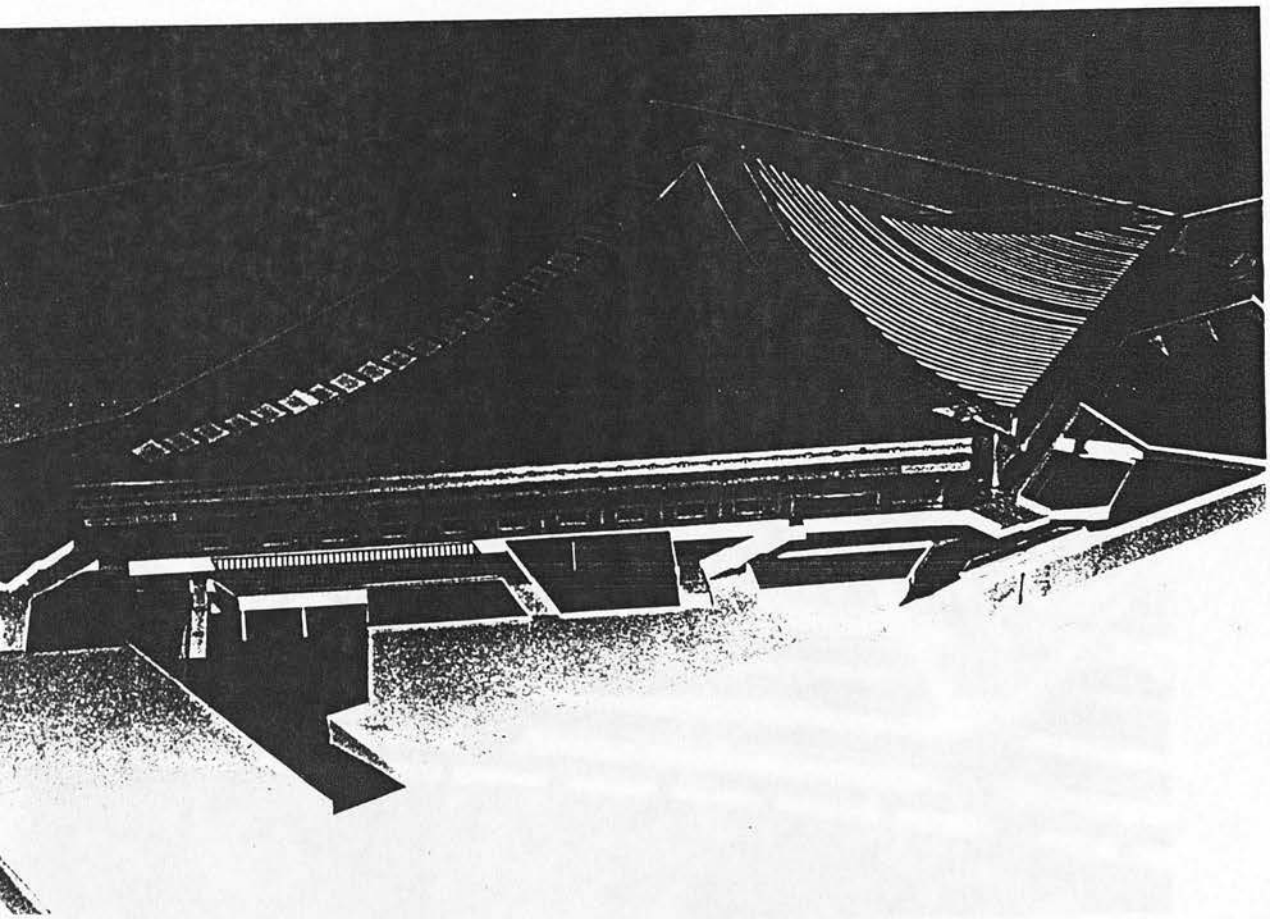


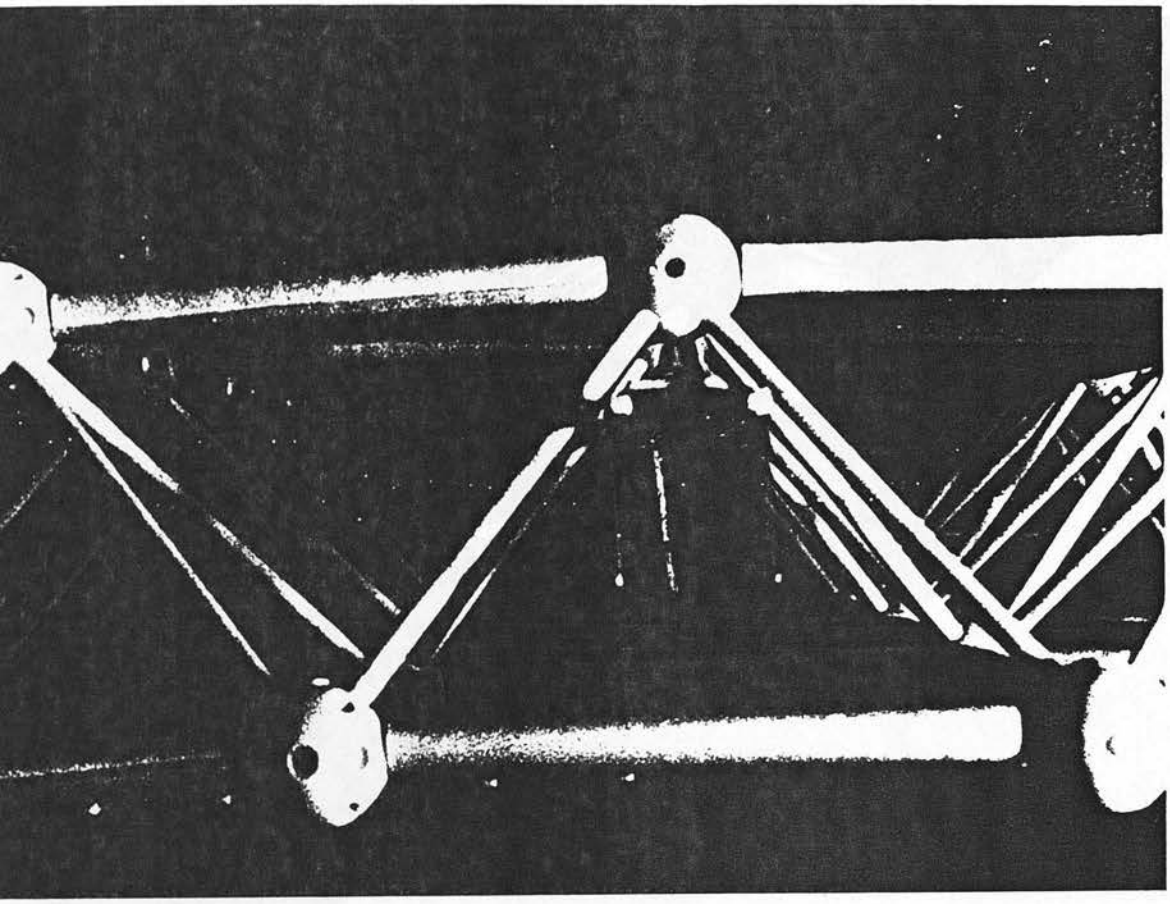


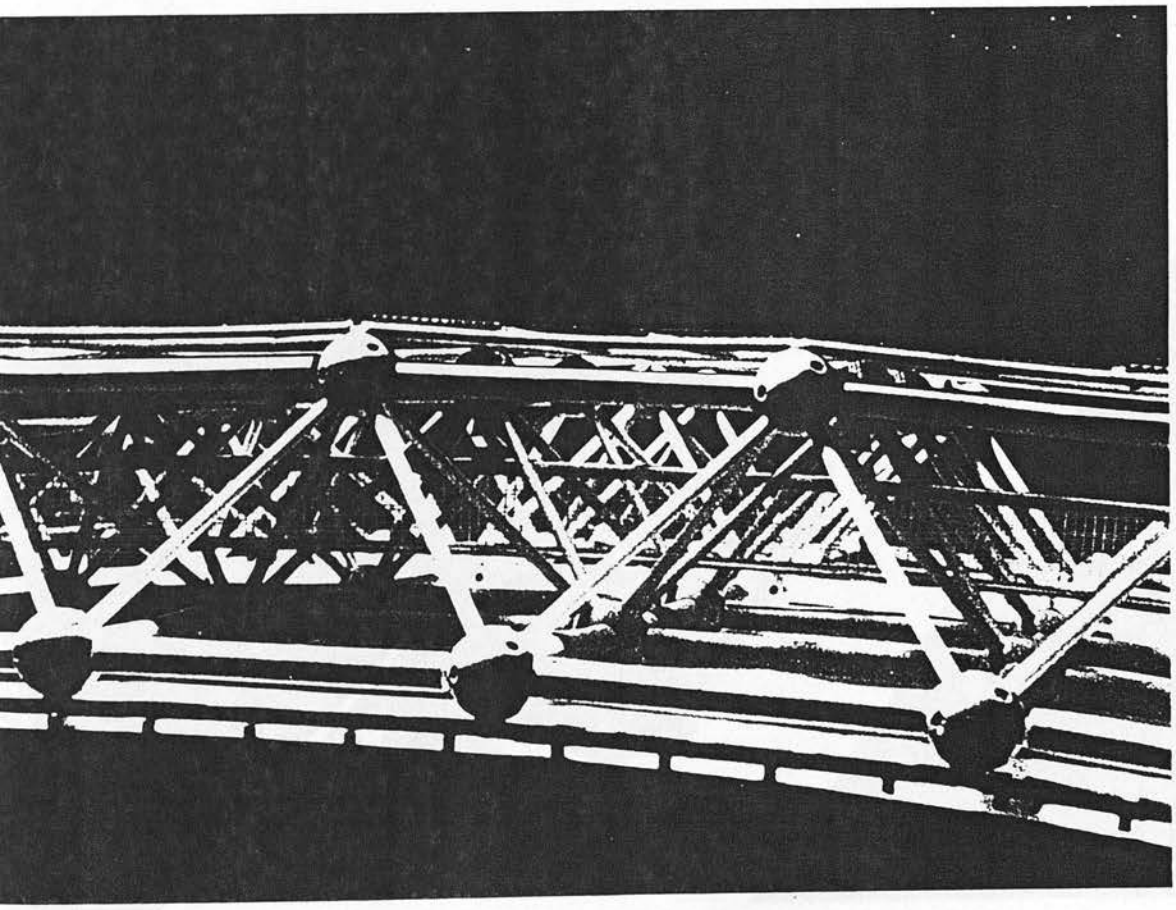


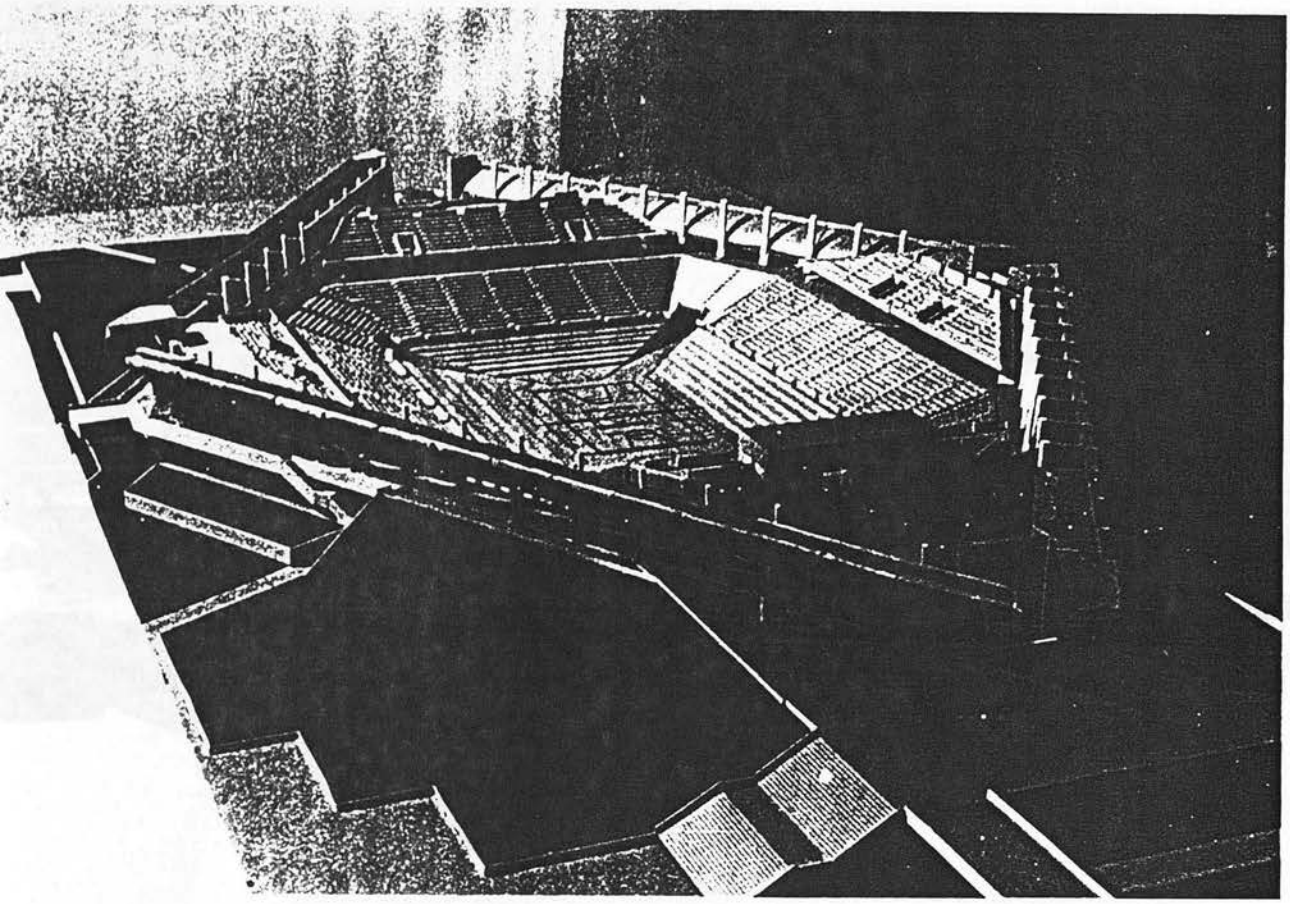


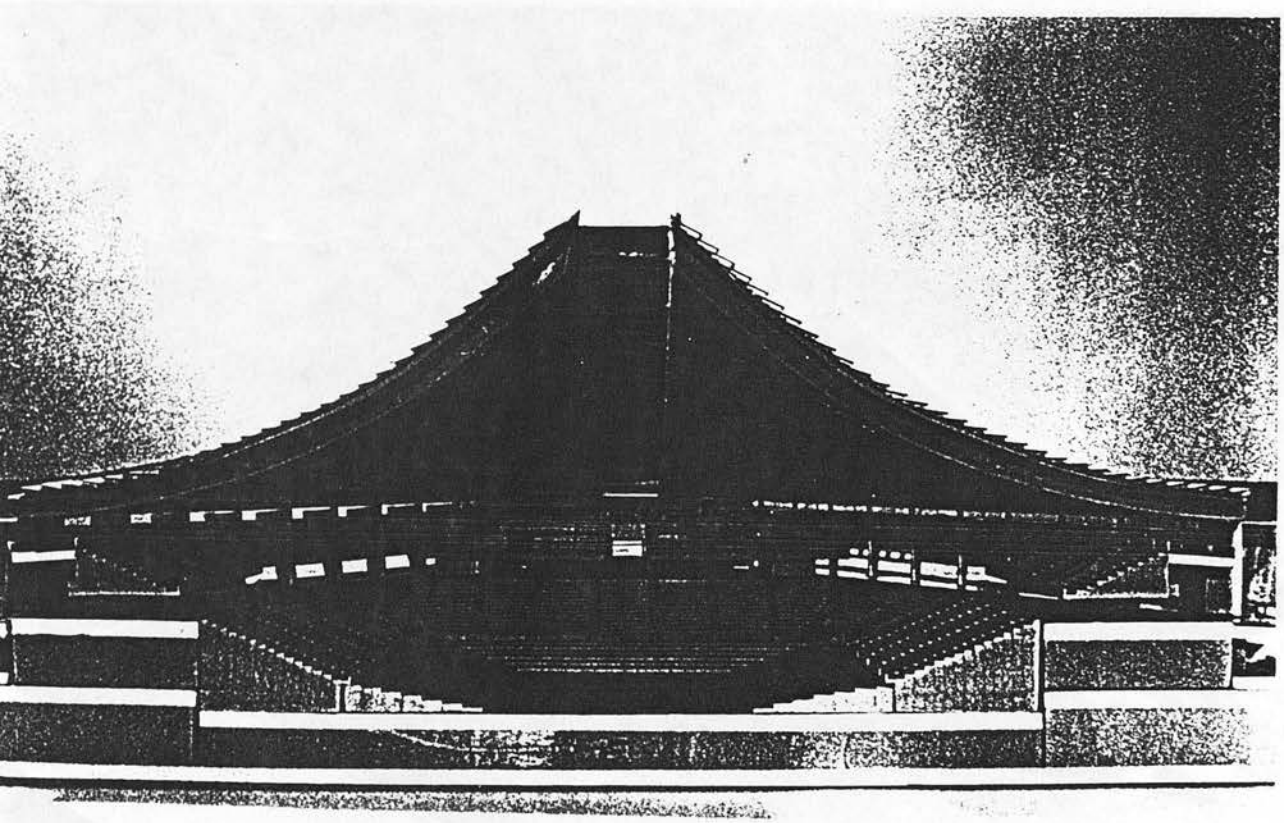


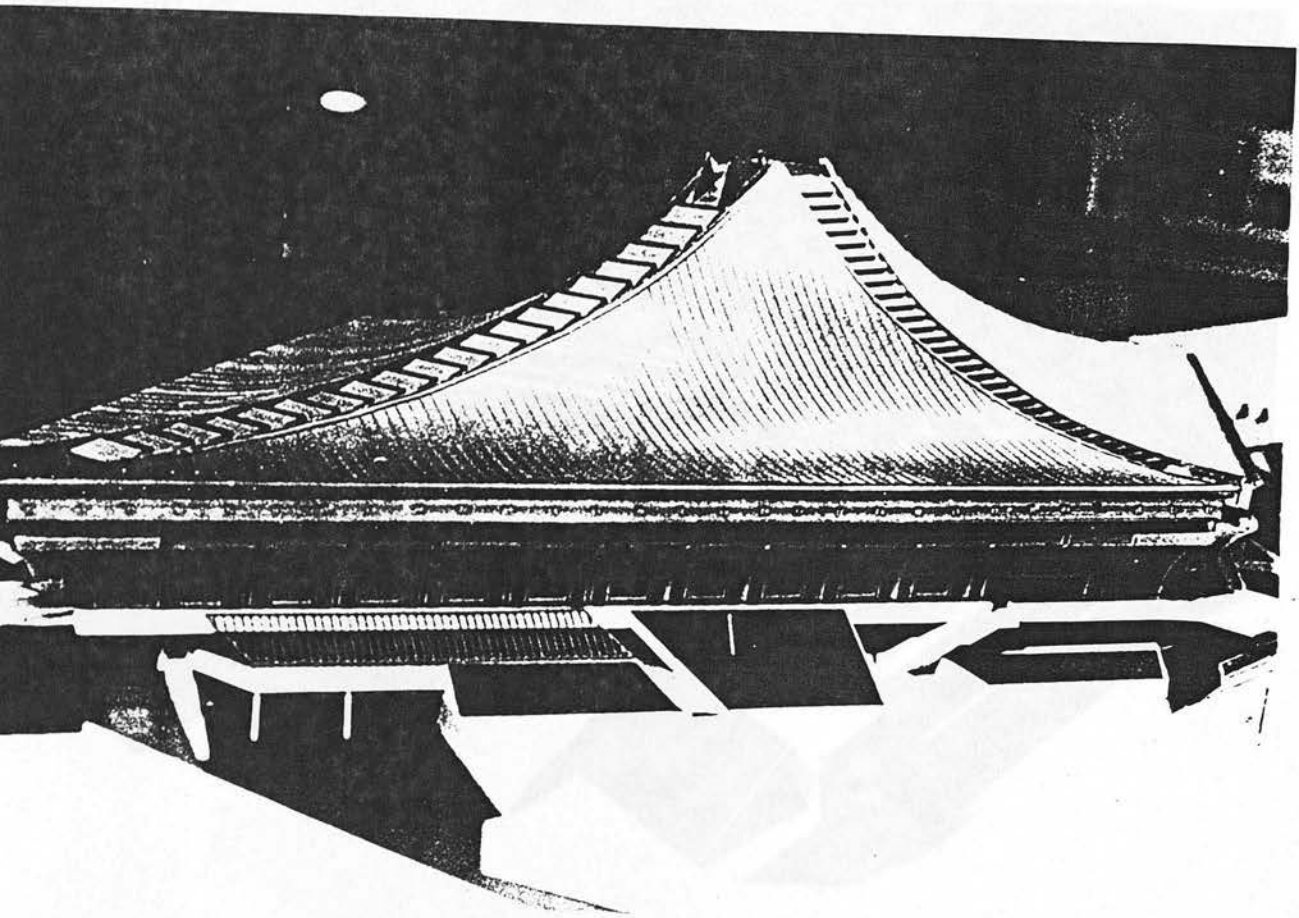


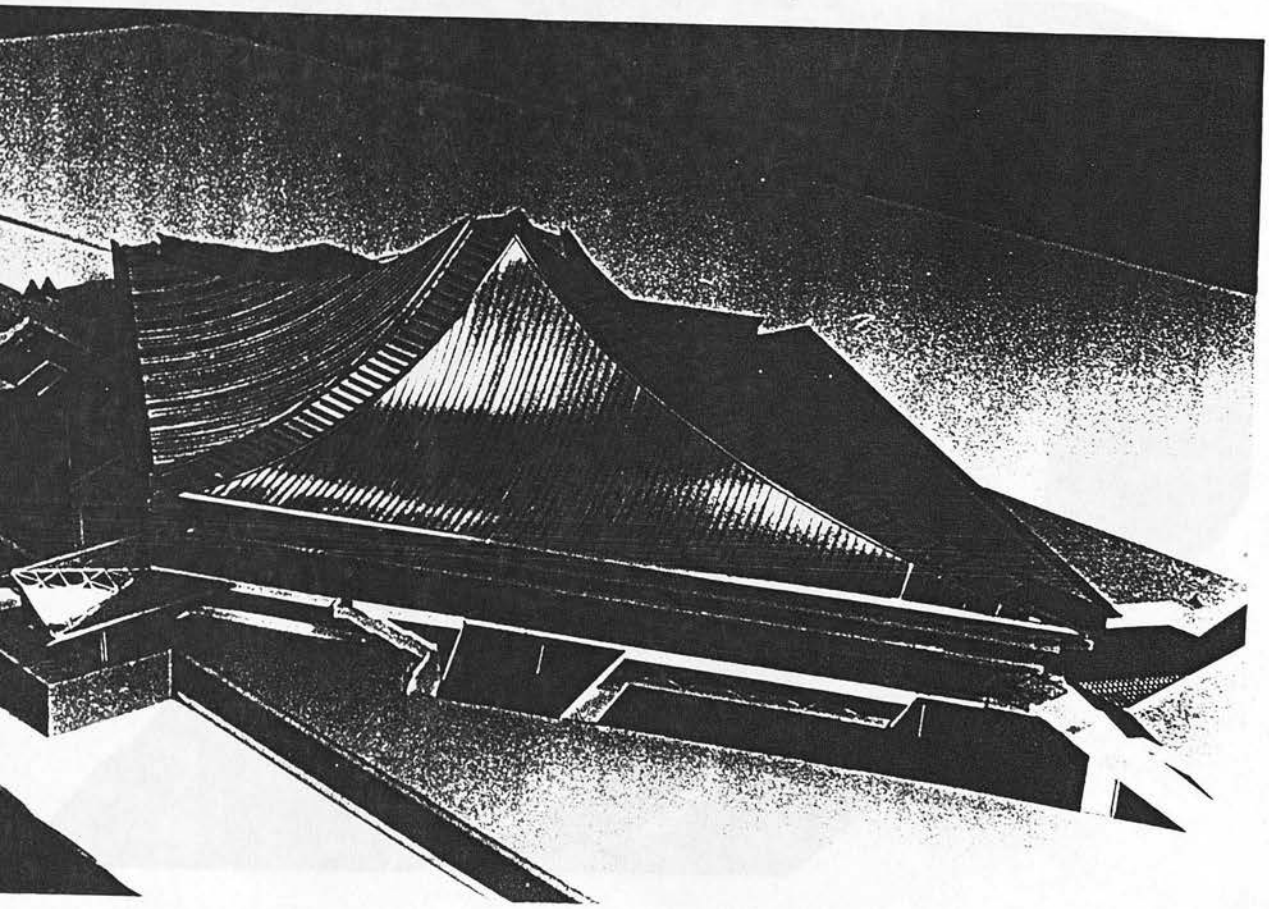


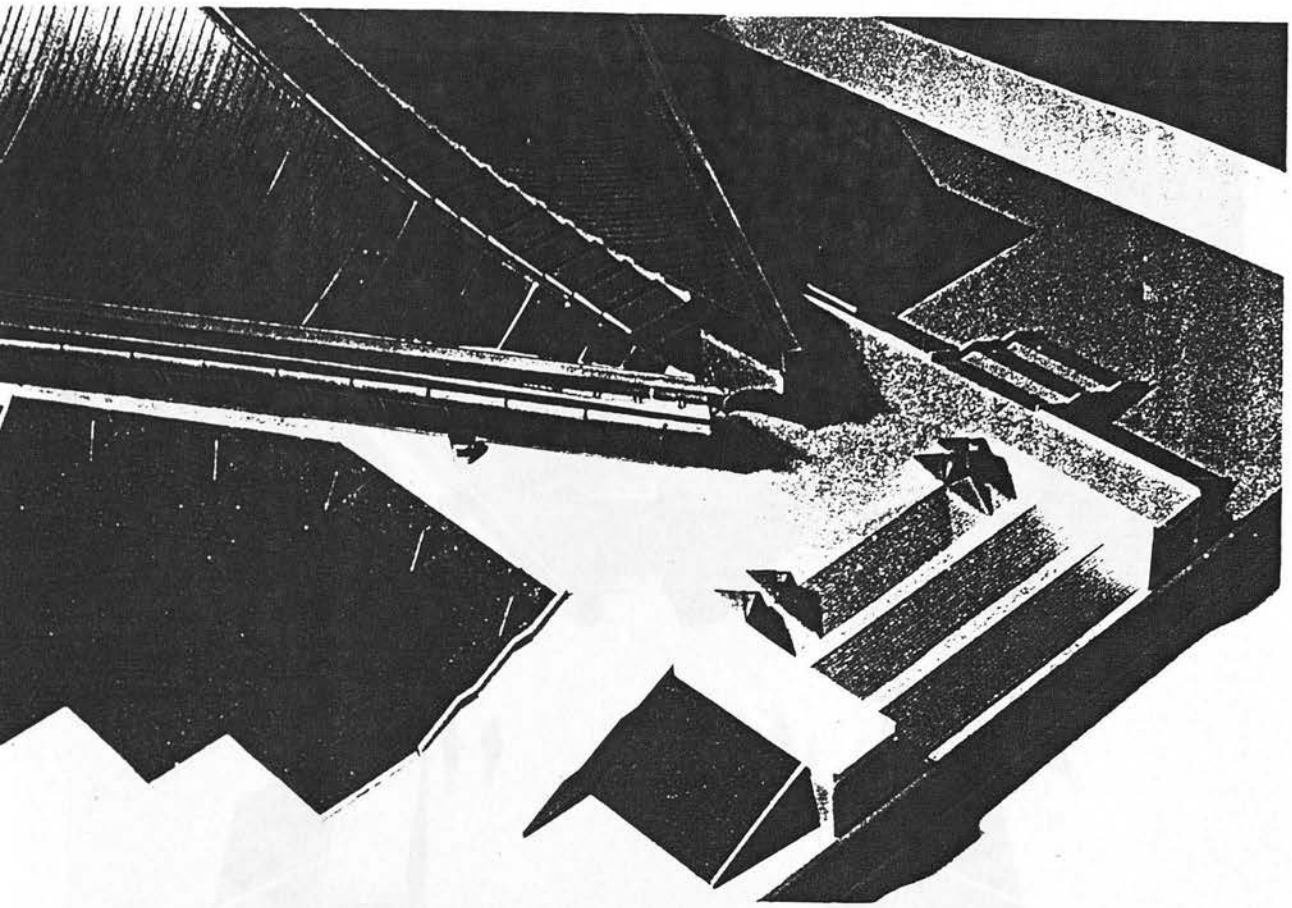


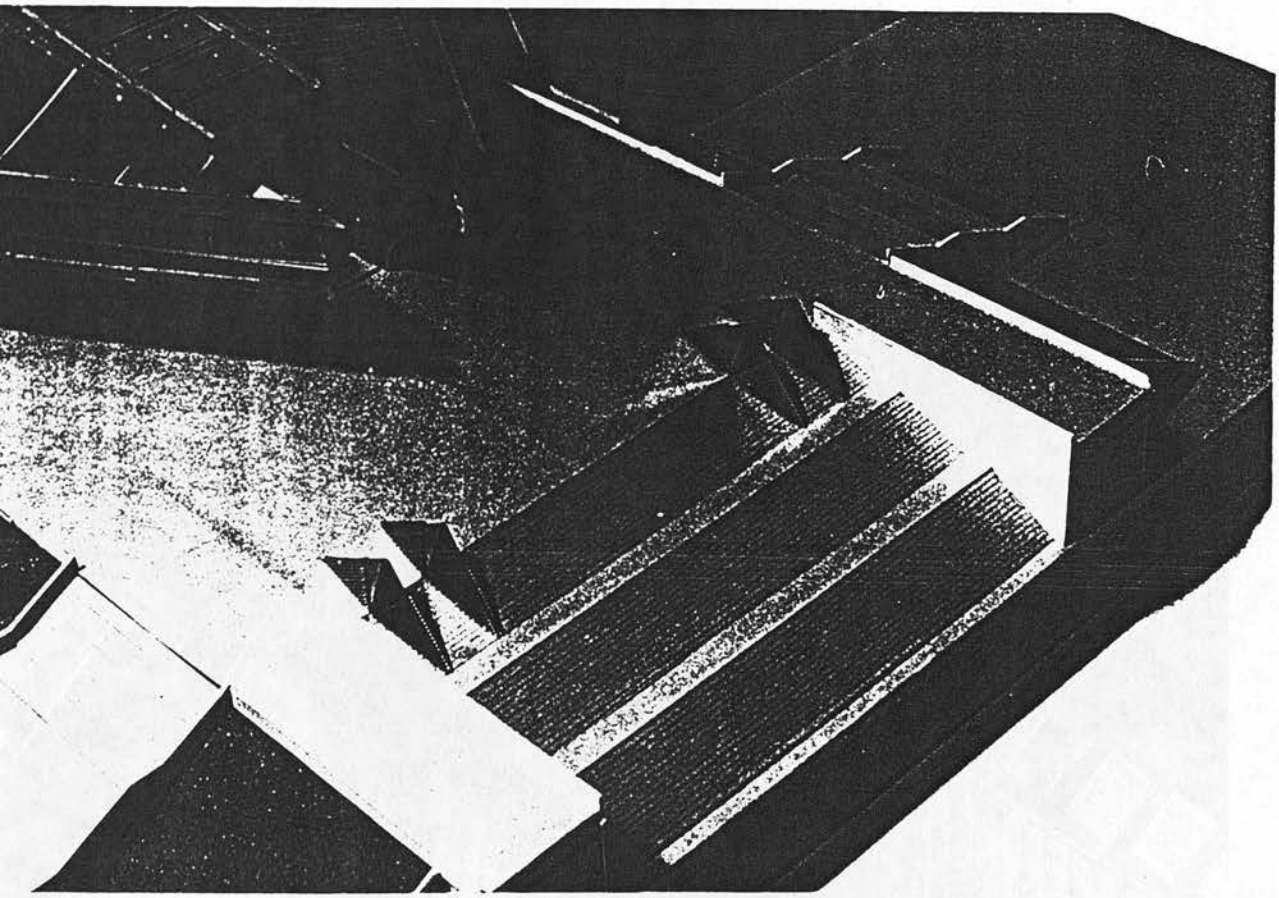


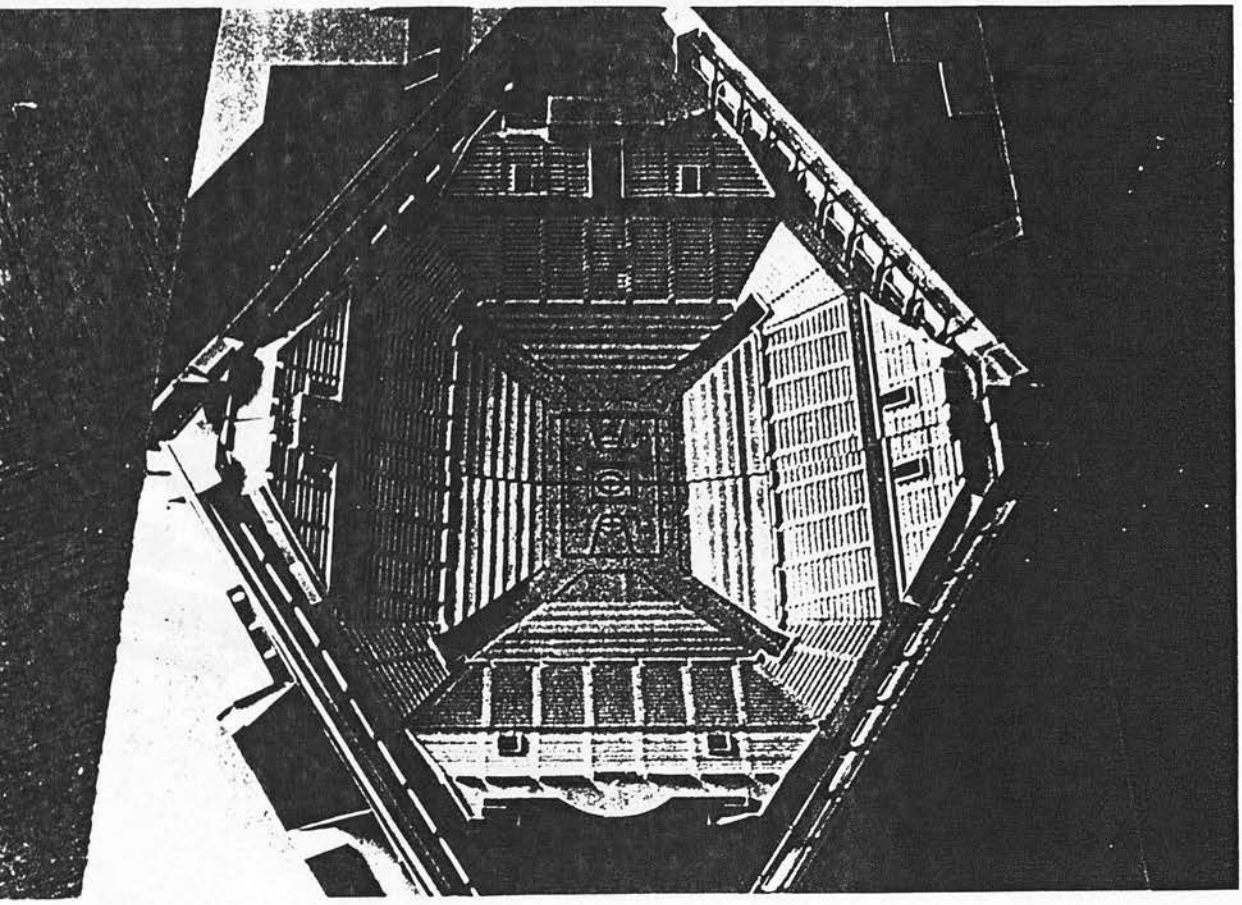


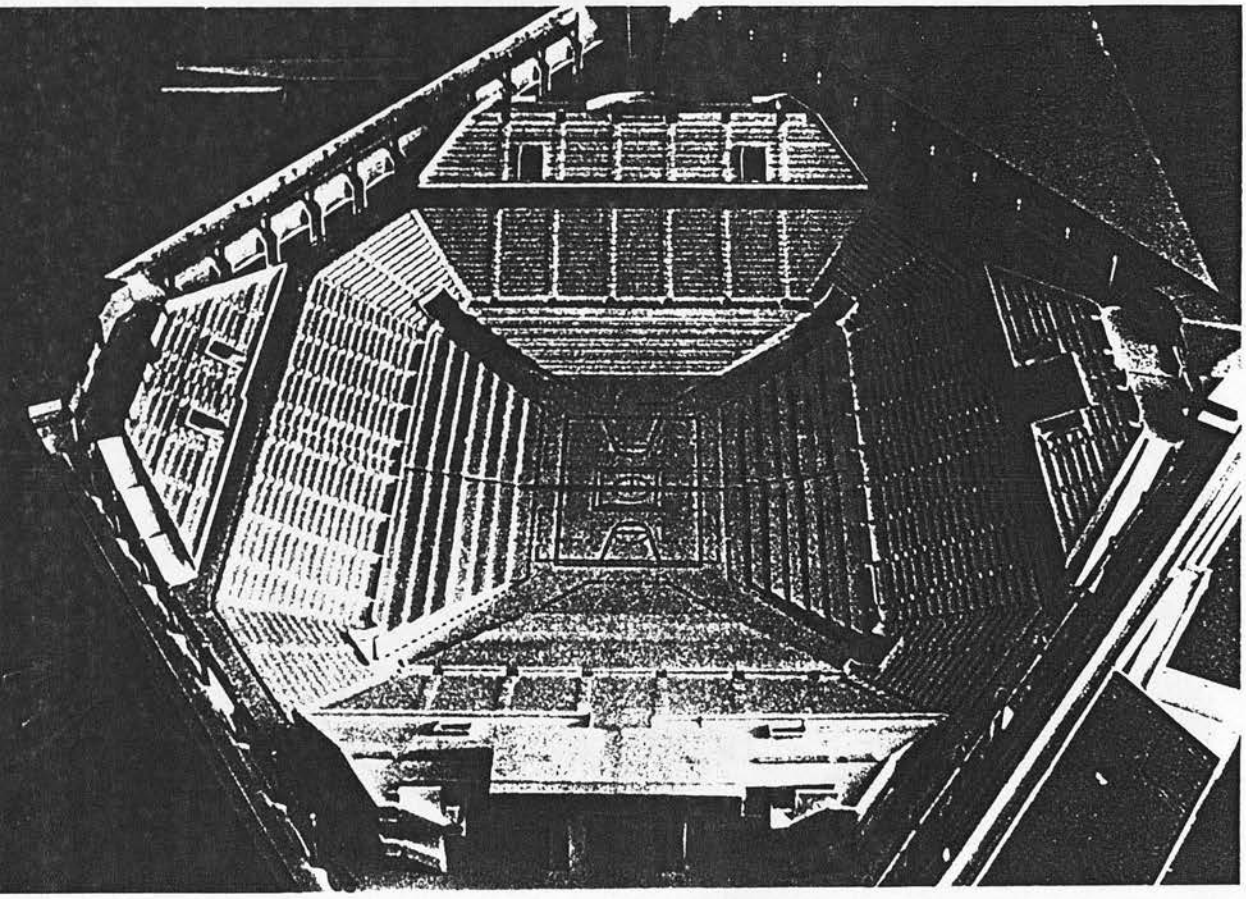


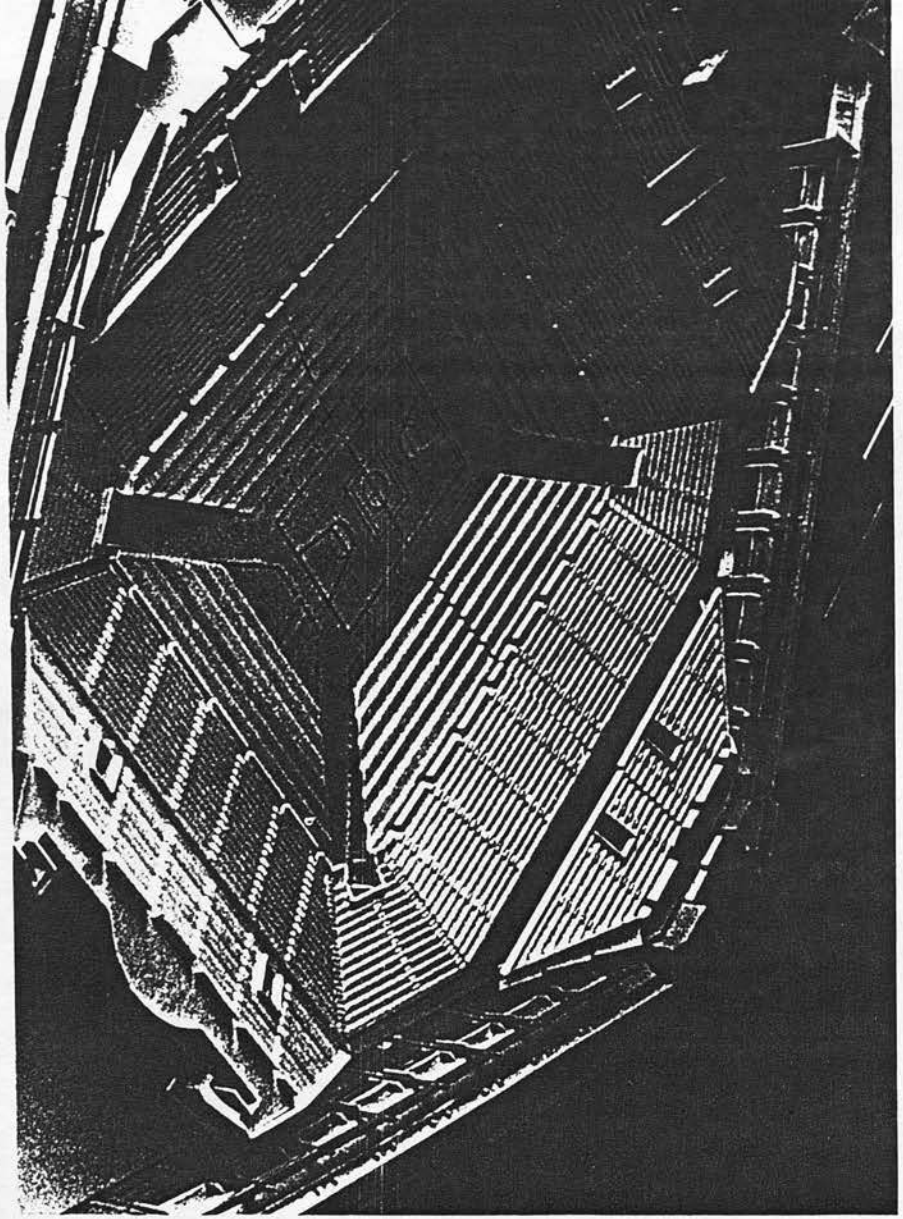


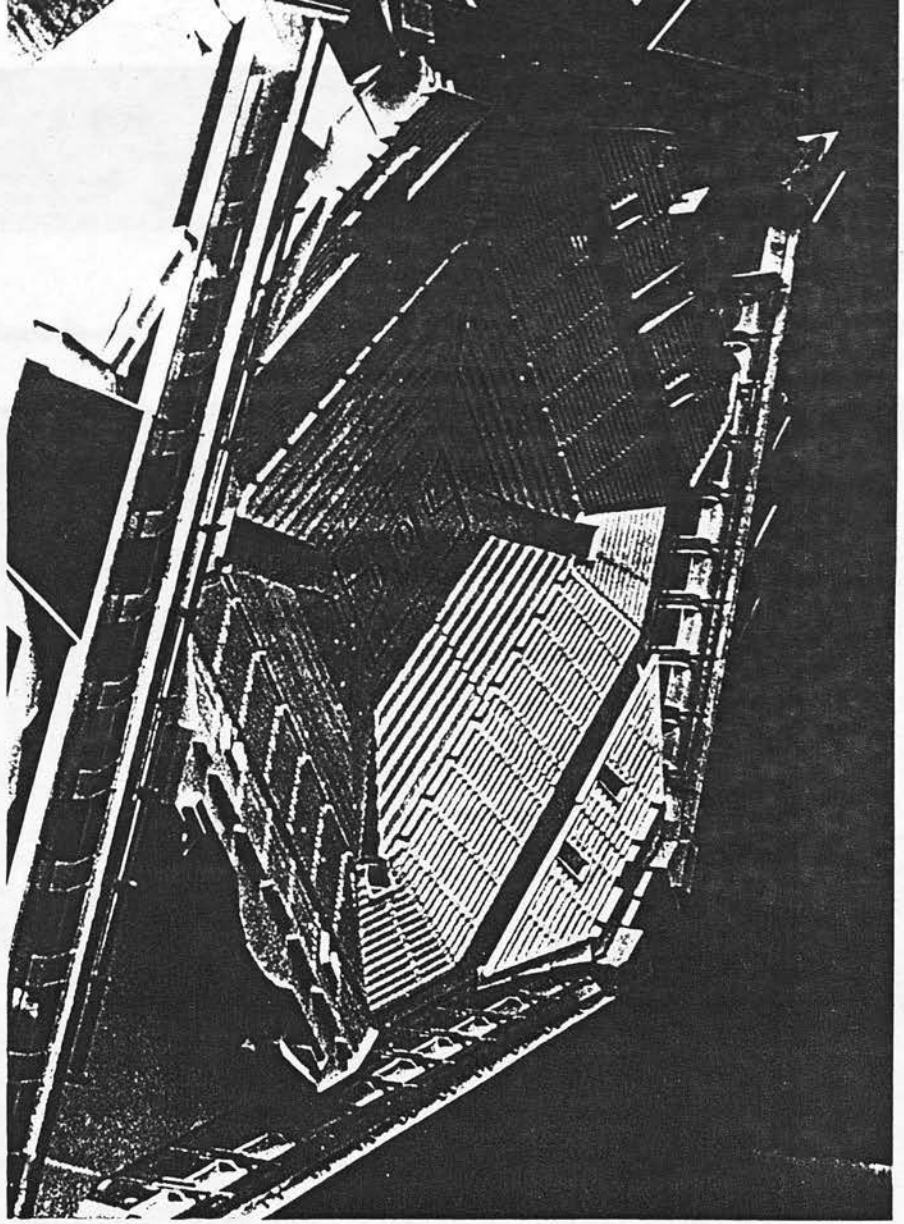


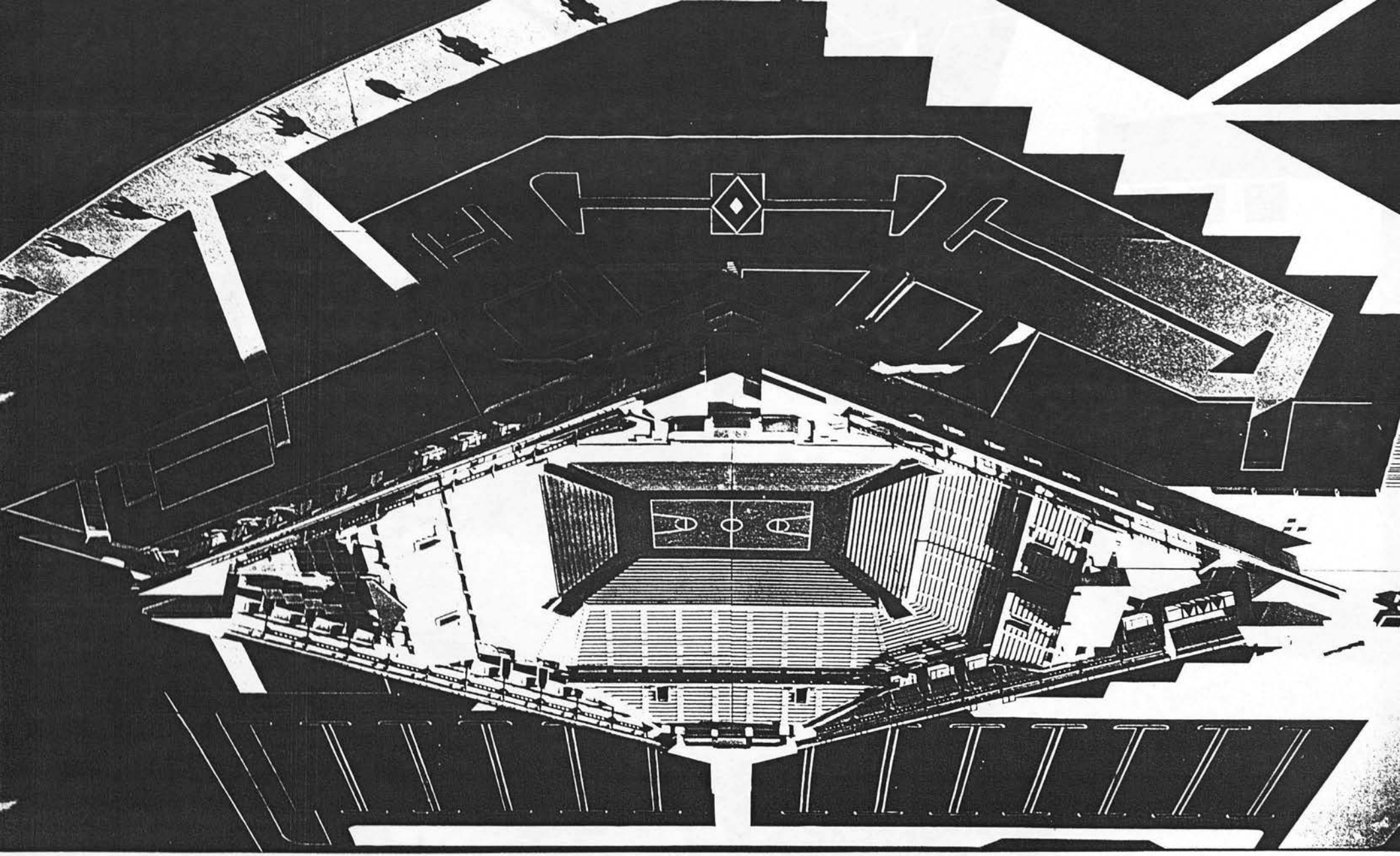


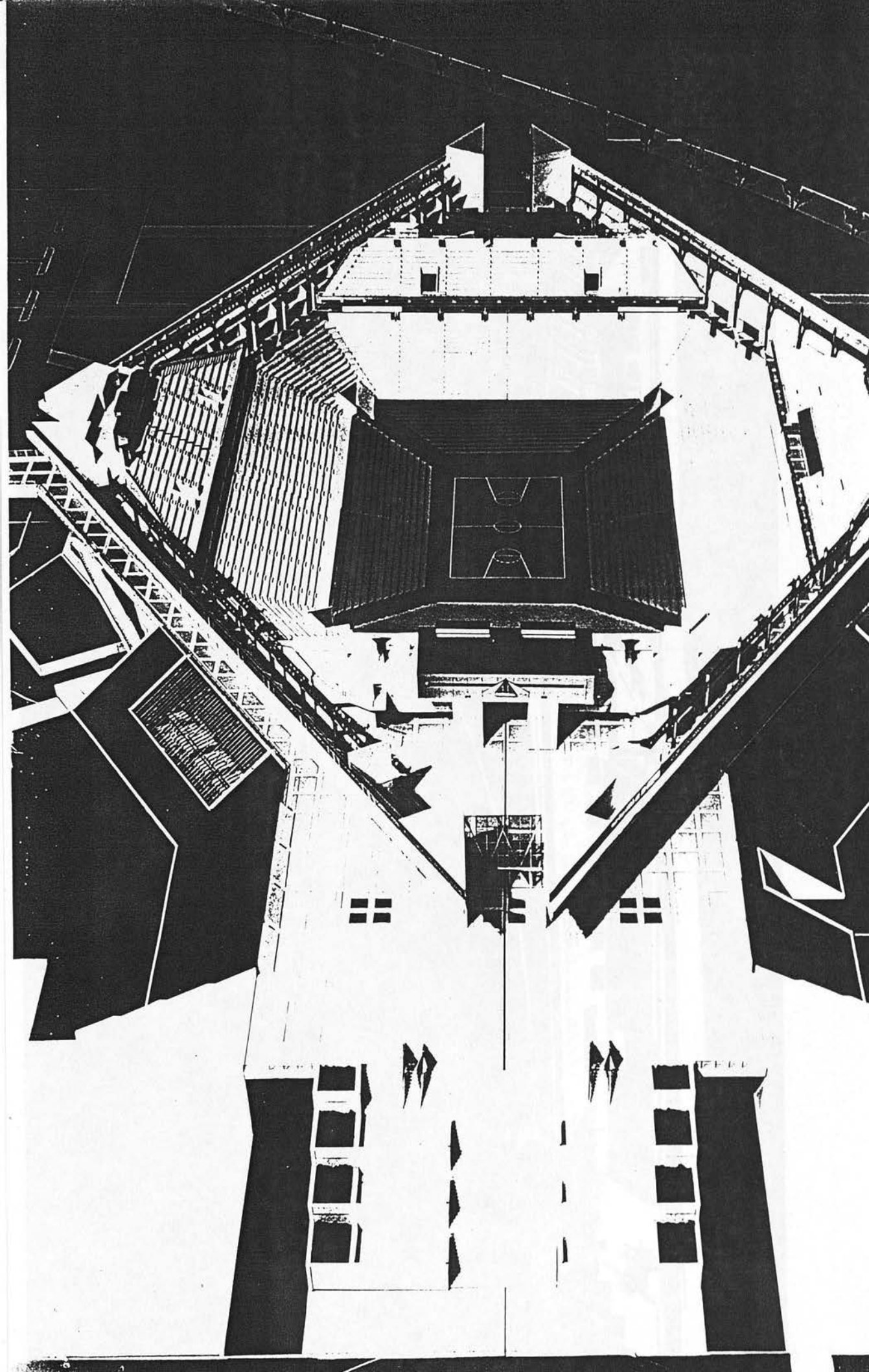


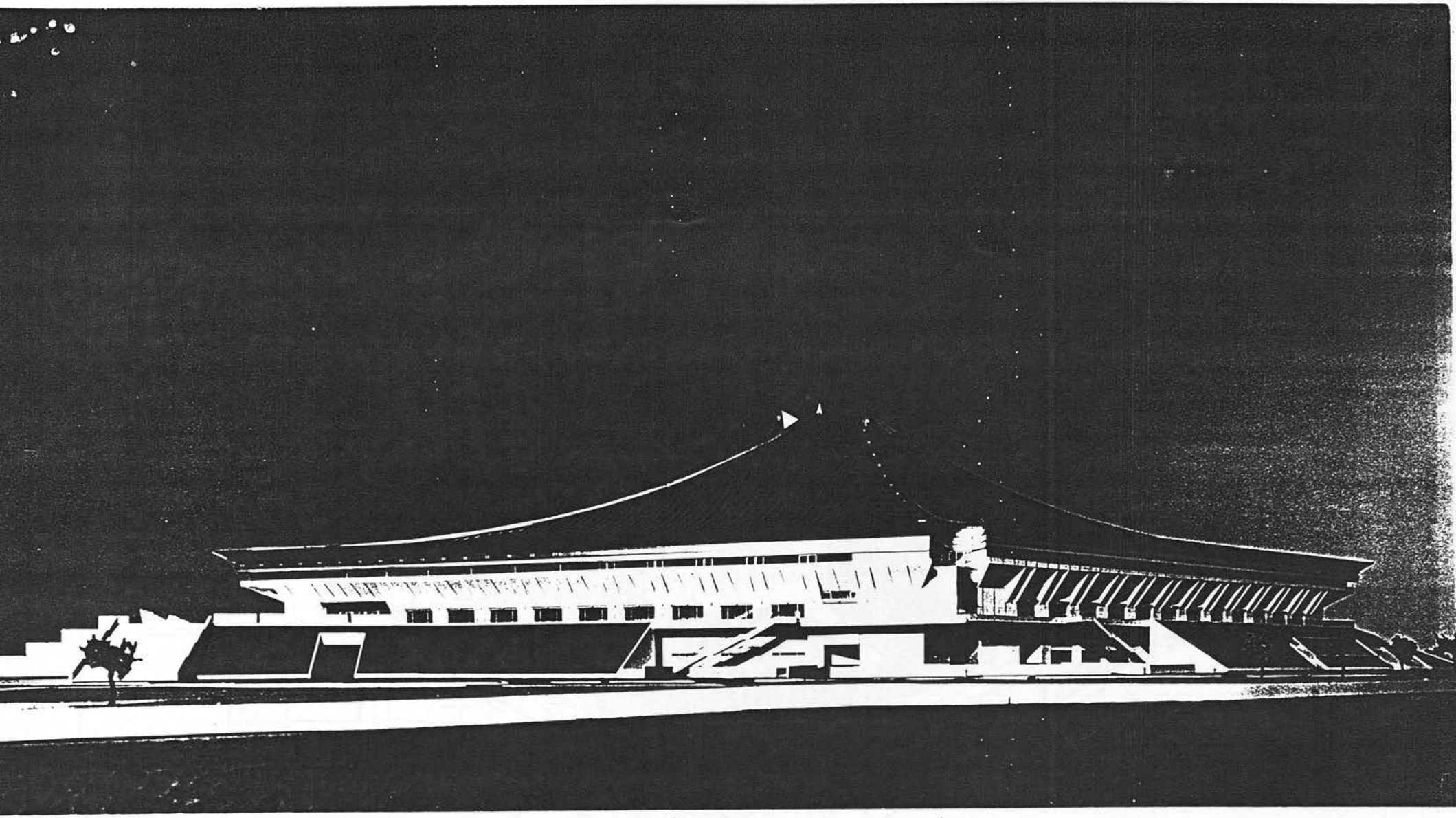


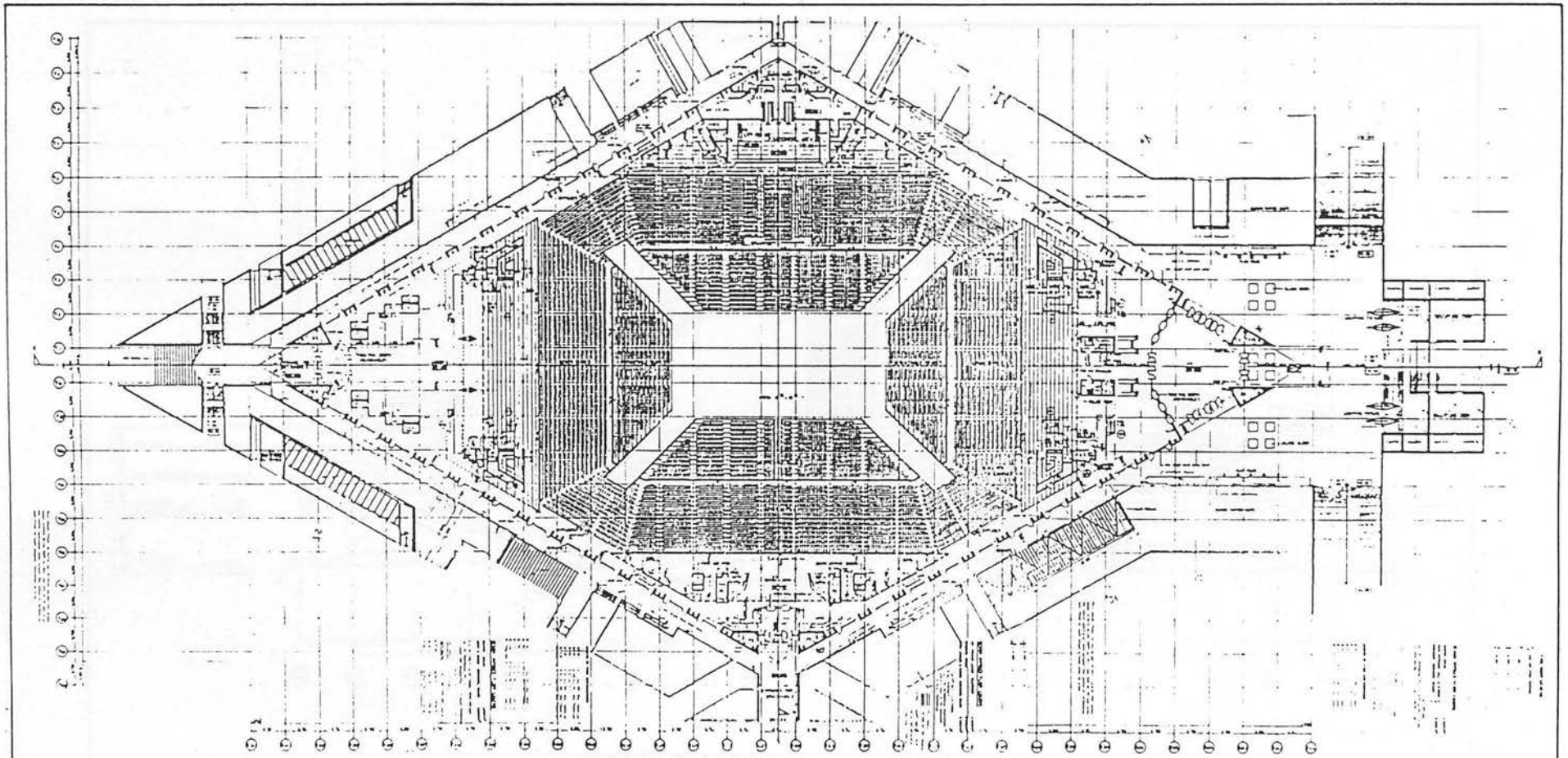




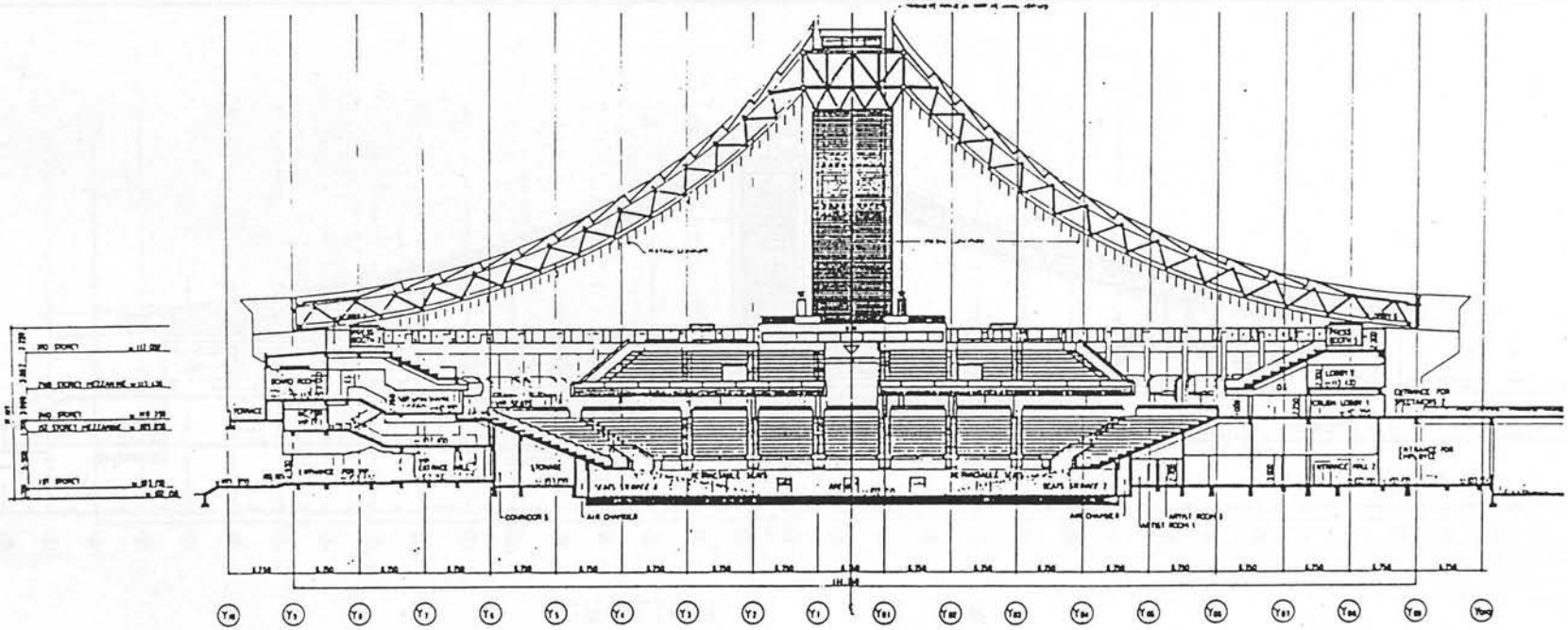




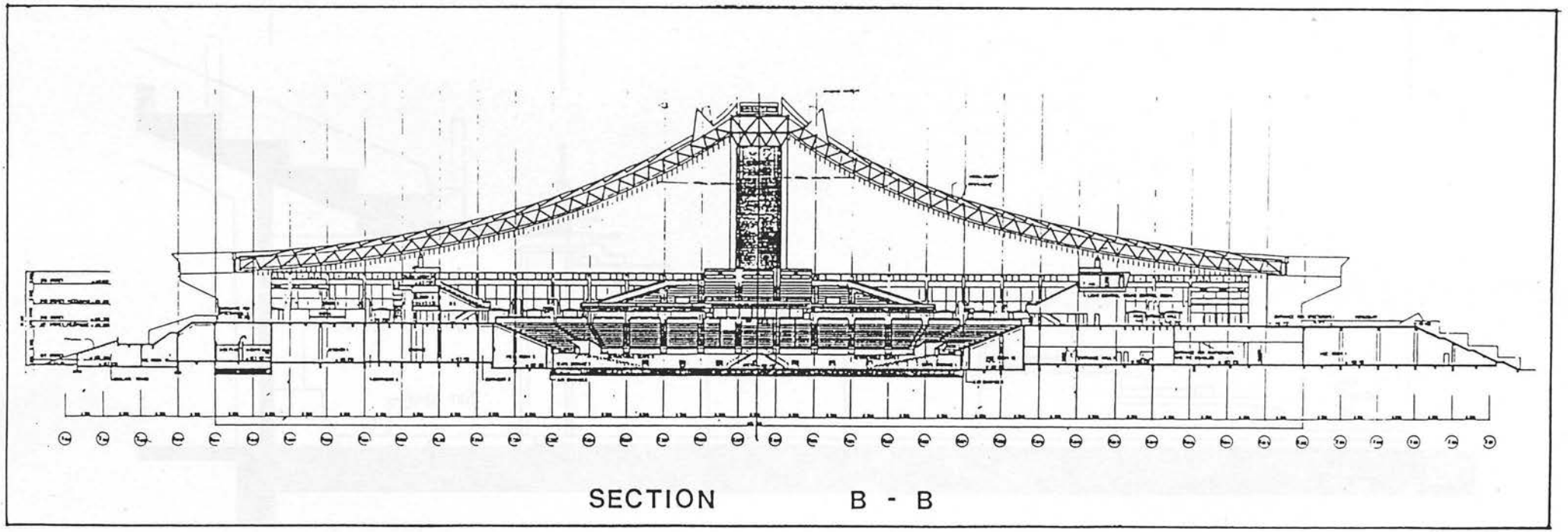




2 ND STOREY PLAN



SECTION A - A



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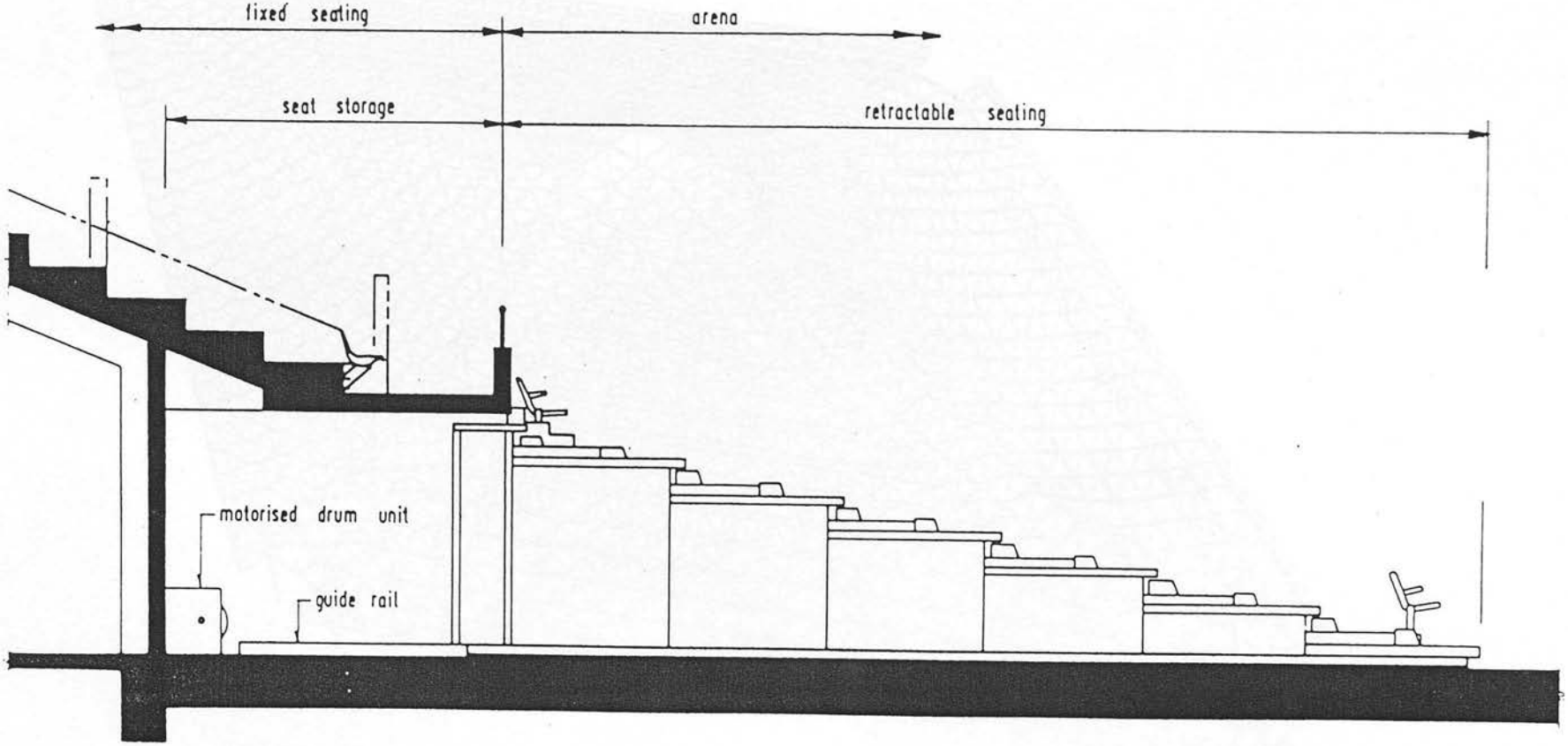
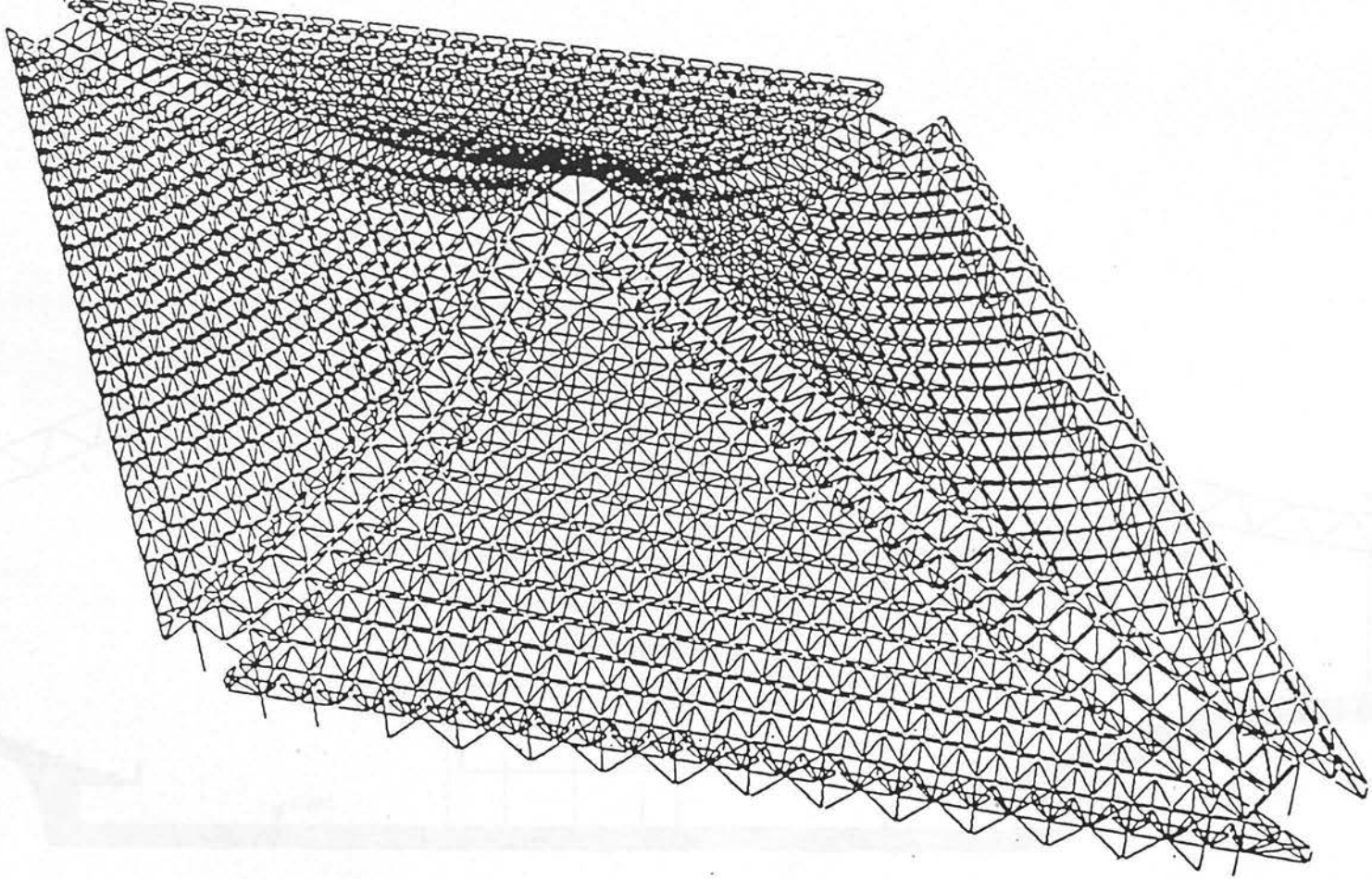
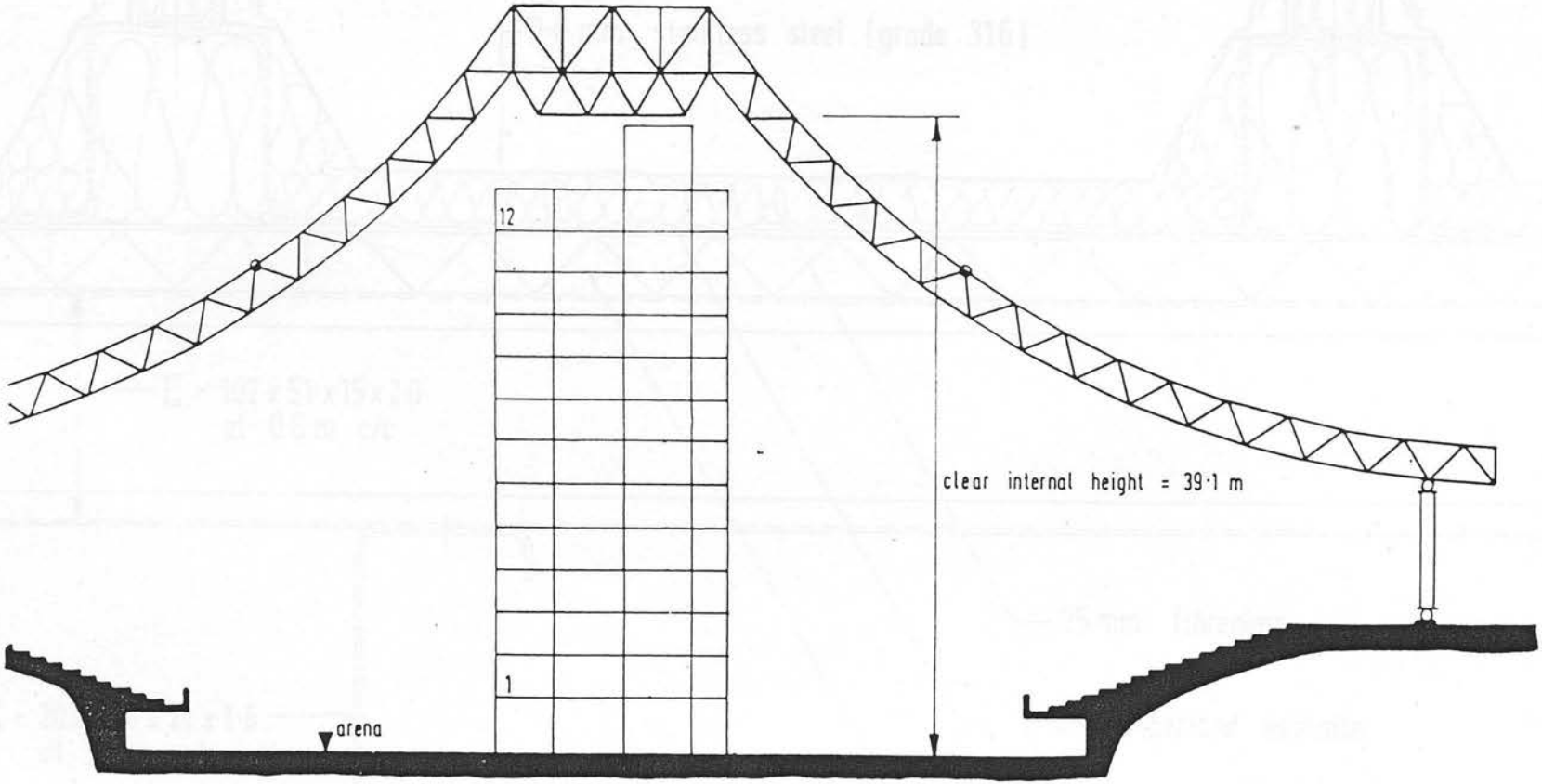


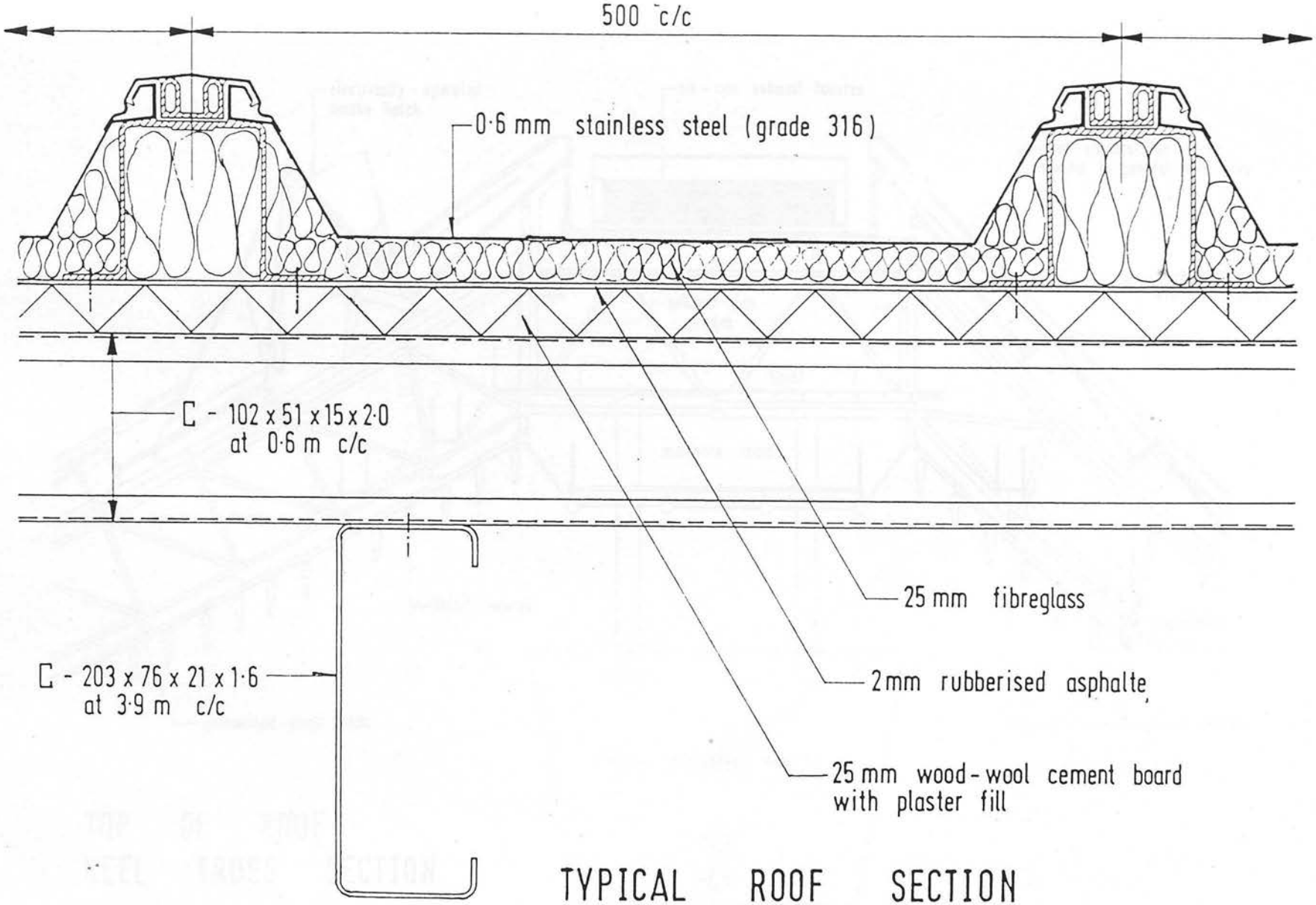
RETRACTABLE SEATING - TYPICAL SECTION



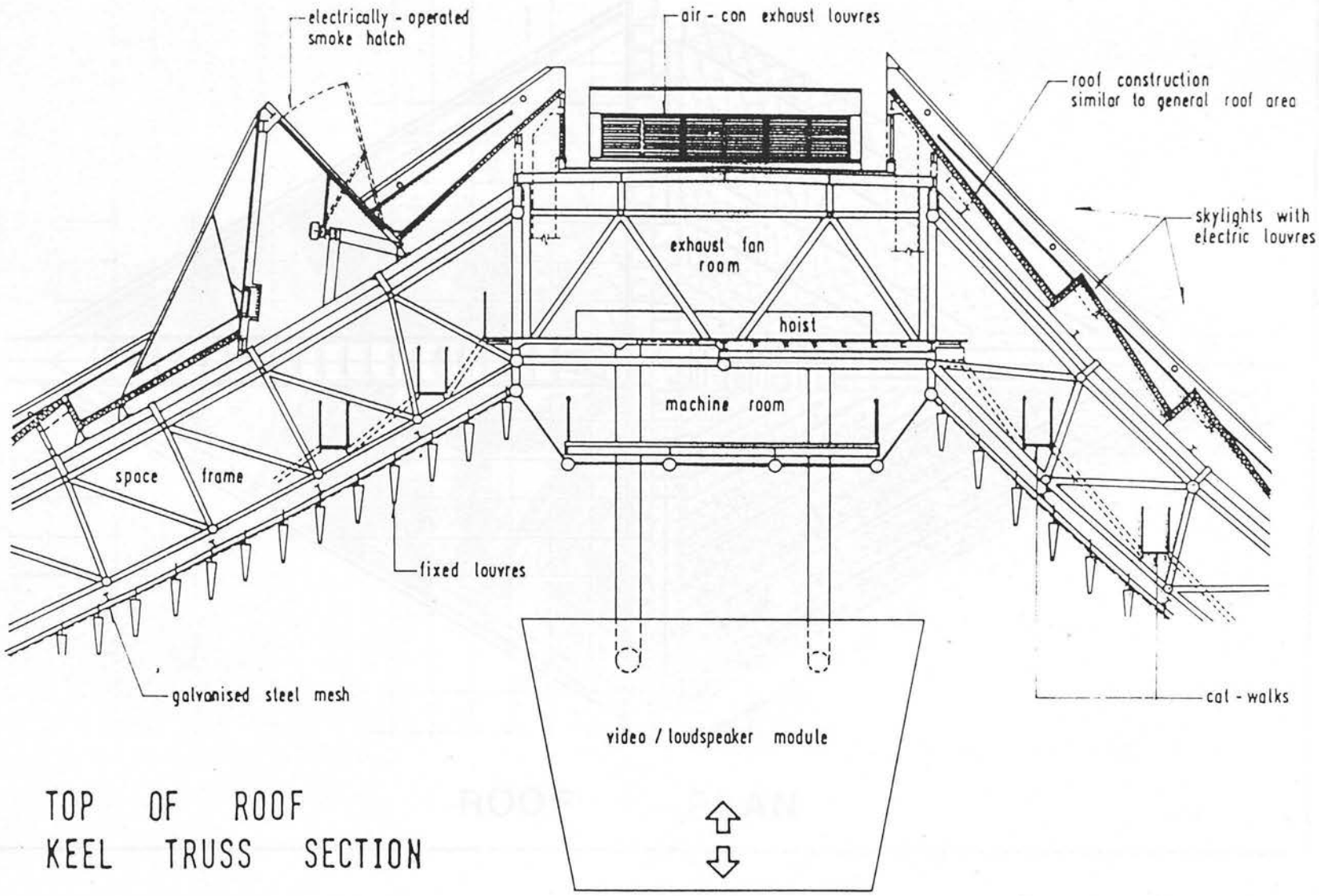
SPACE - FRAME : ISOMETRIC VIEW



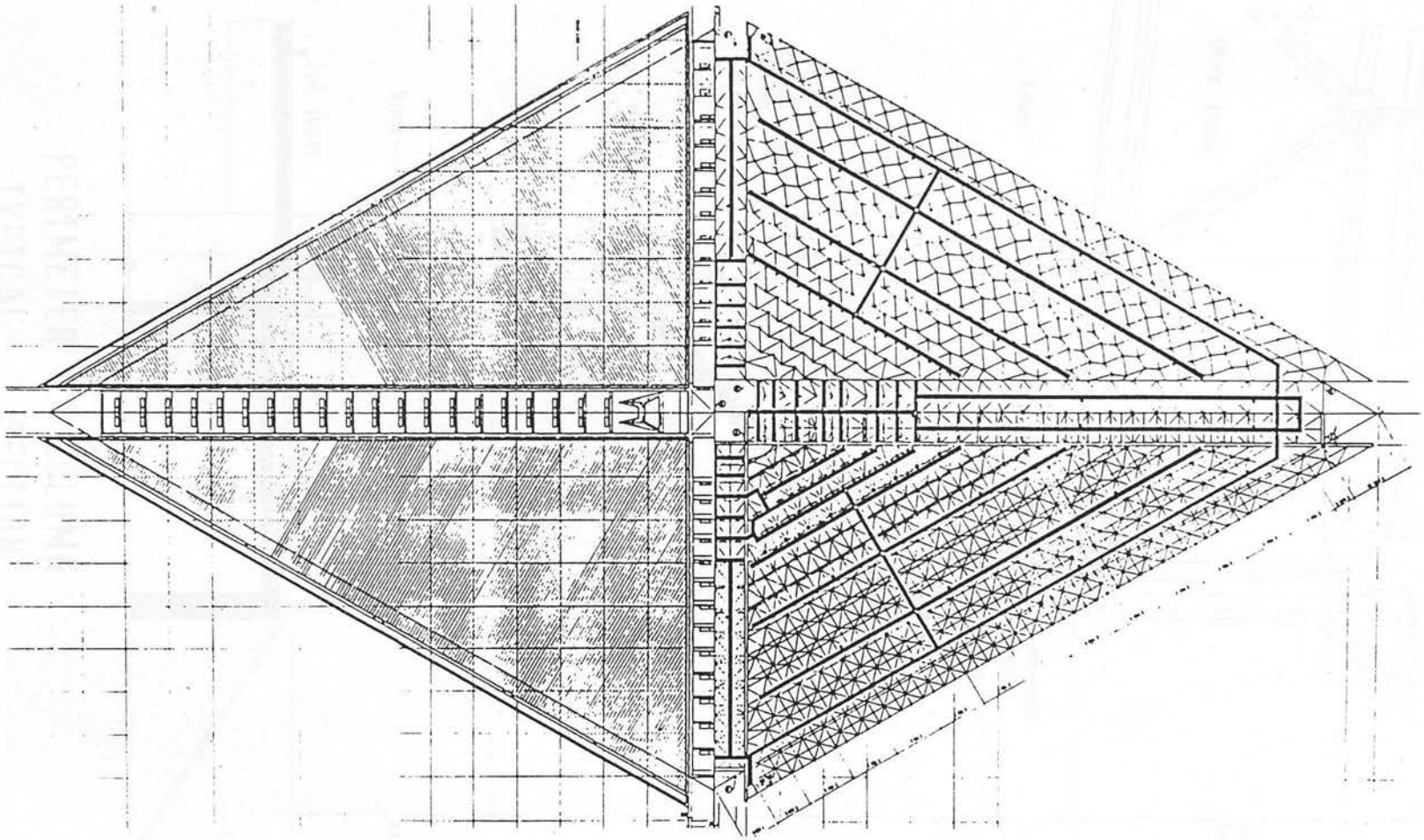
RELATIVE HEIGHT OF 12 - STOREY HDB BLOCK



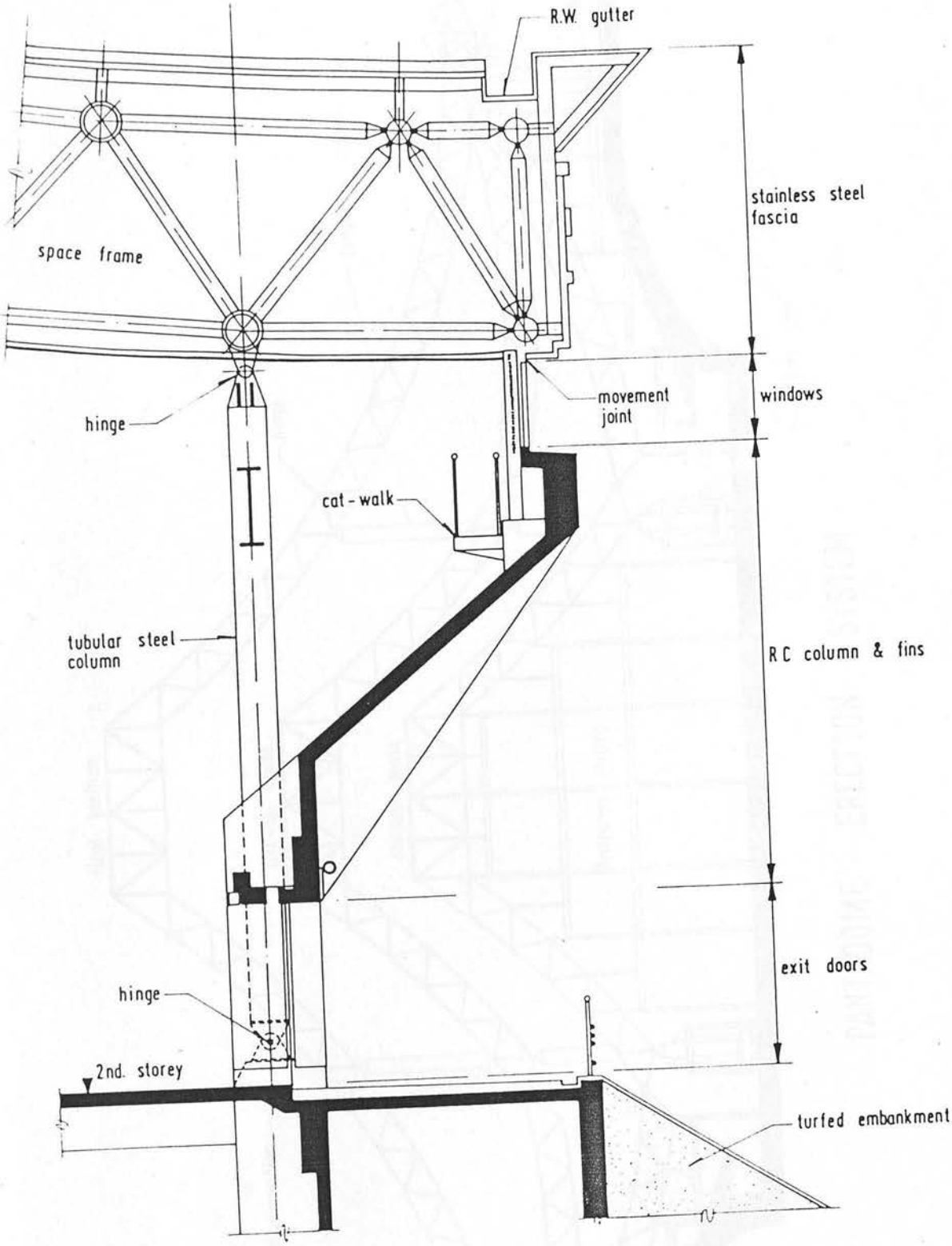
TYPICAL ROOF SECTION



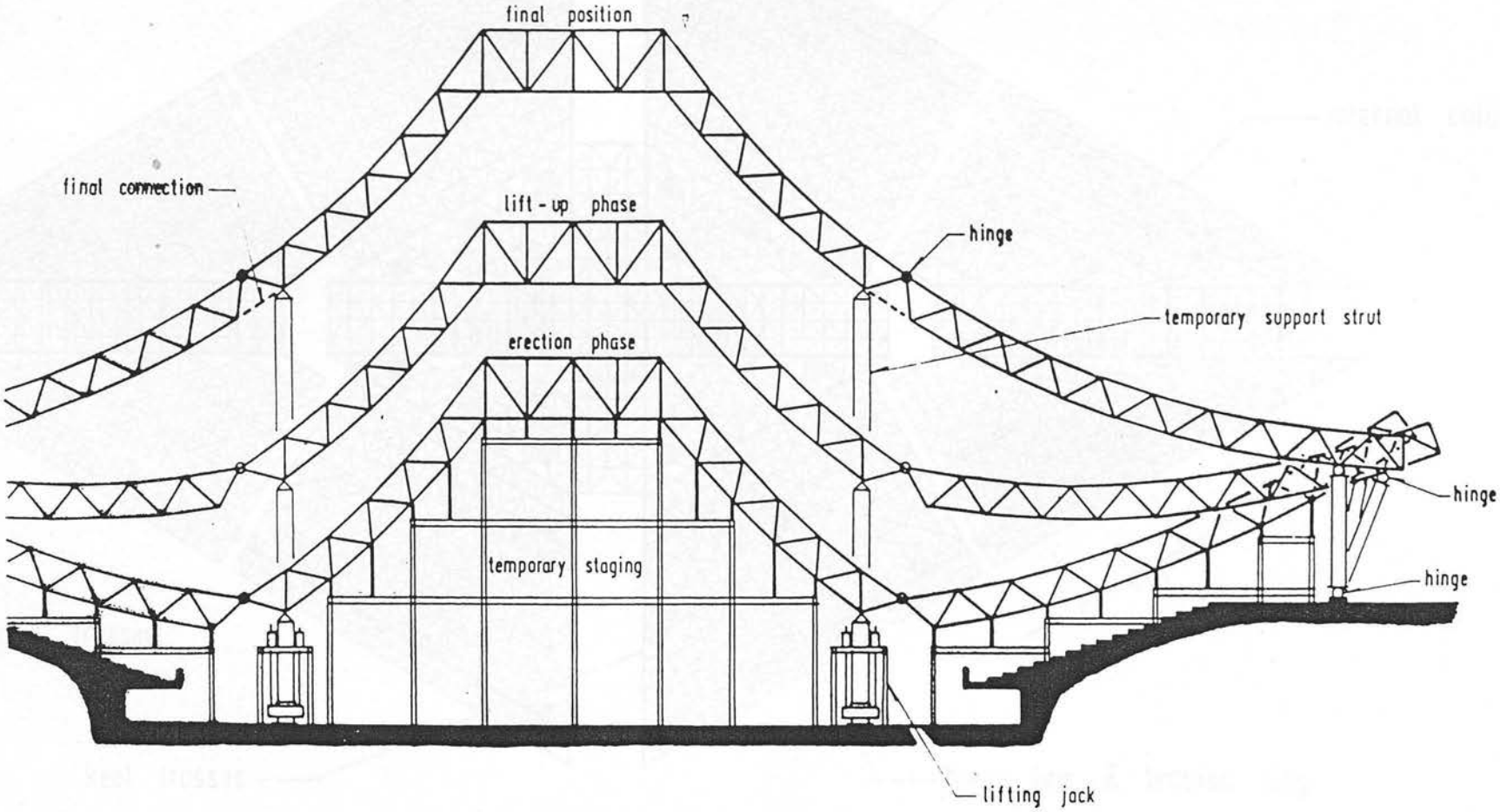
TOP OF ROOF
KEEL TRUSS SECTION



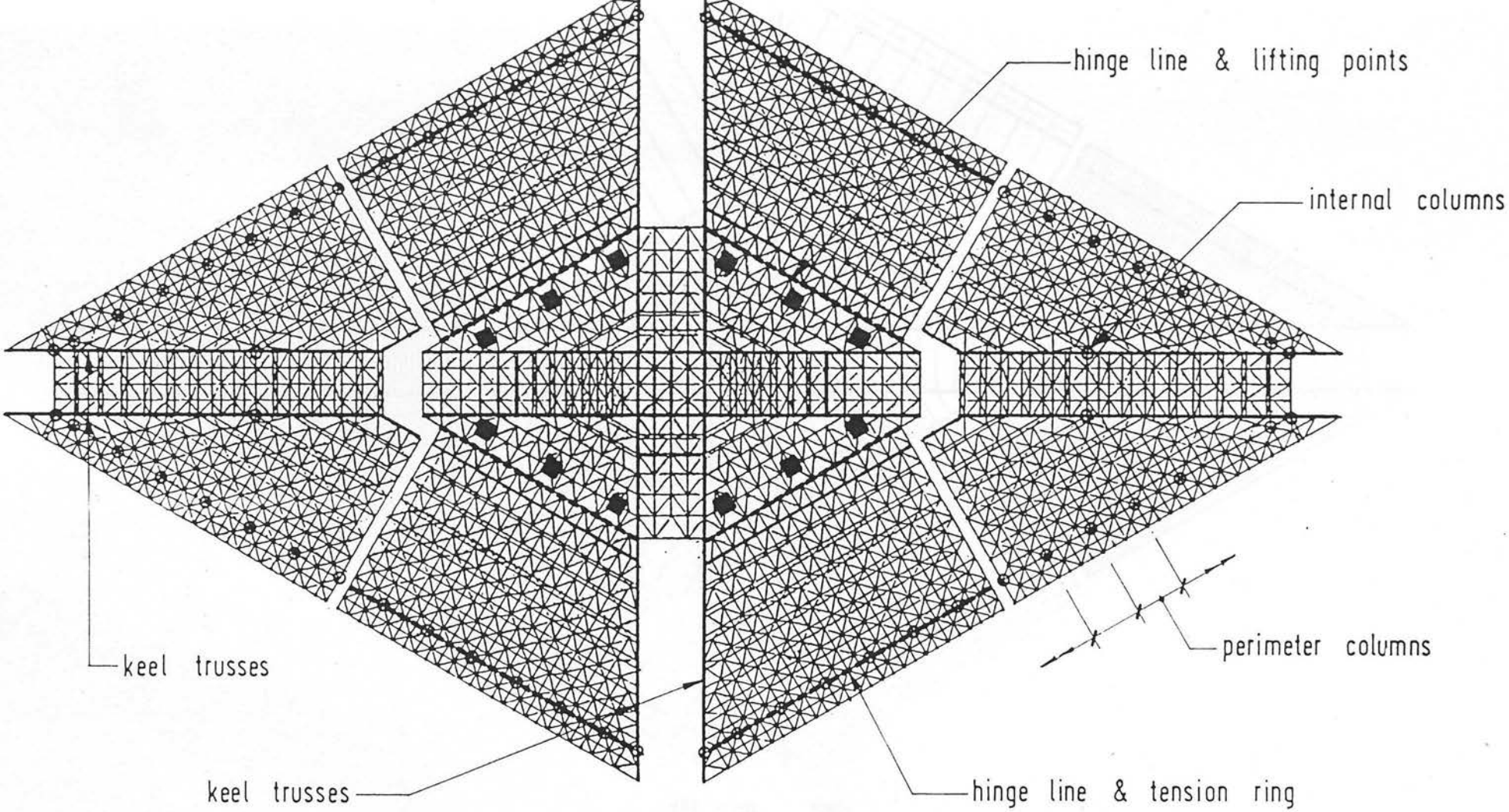
ROOF PLAN



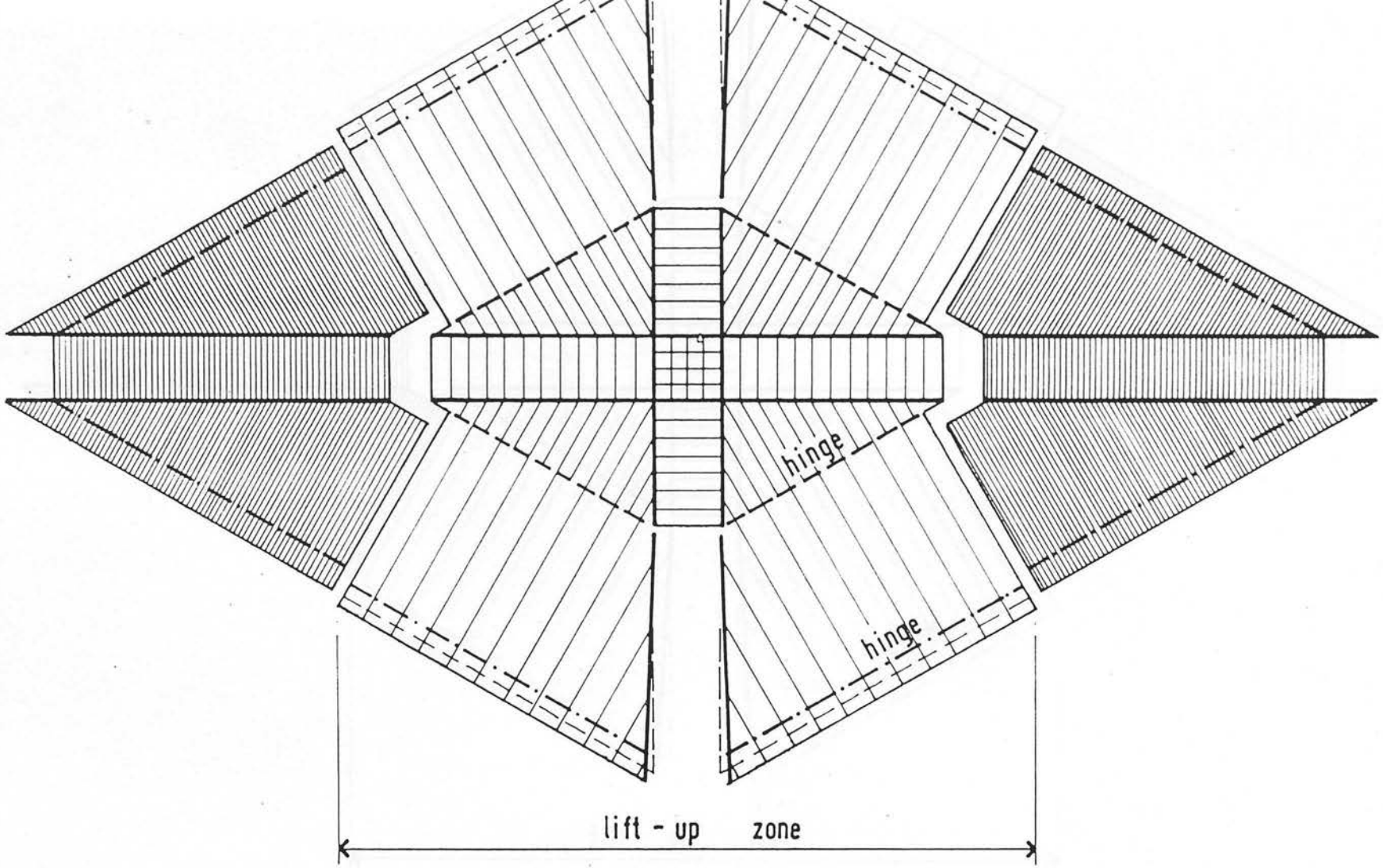
PERIMETER COLUMN
TYPICAL SECTION



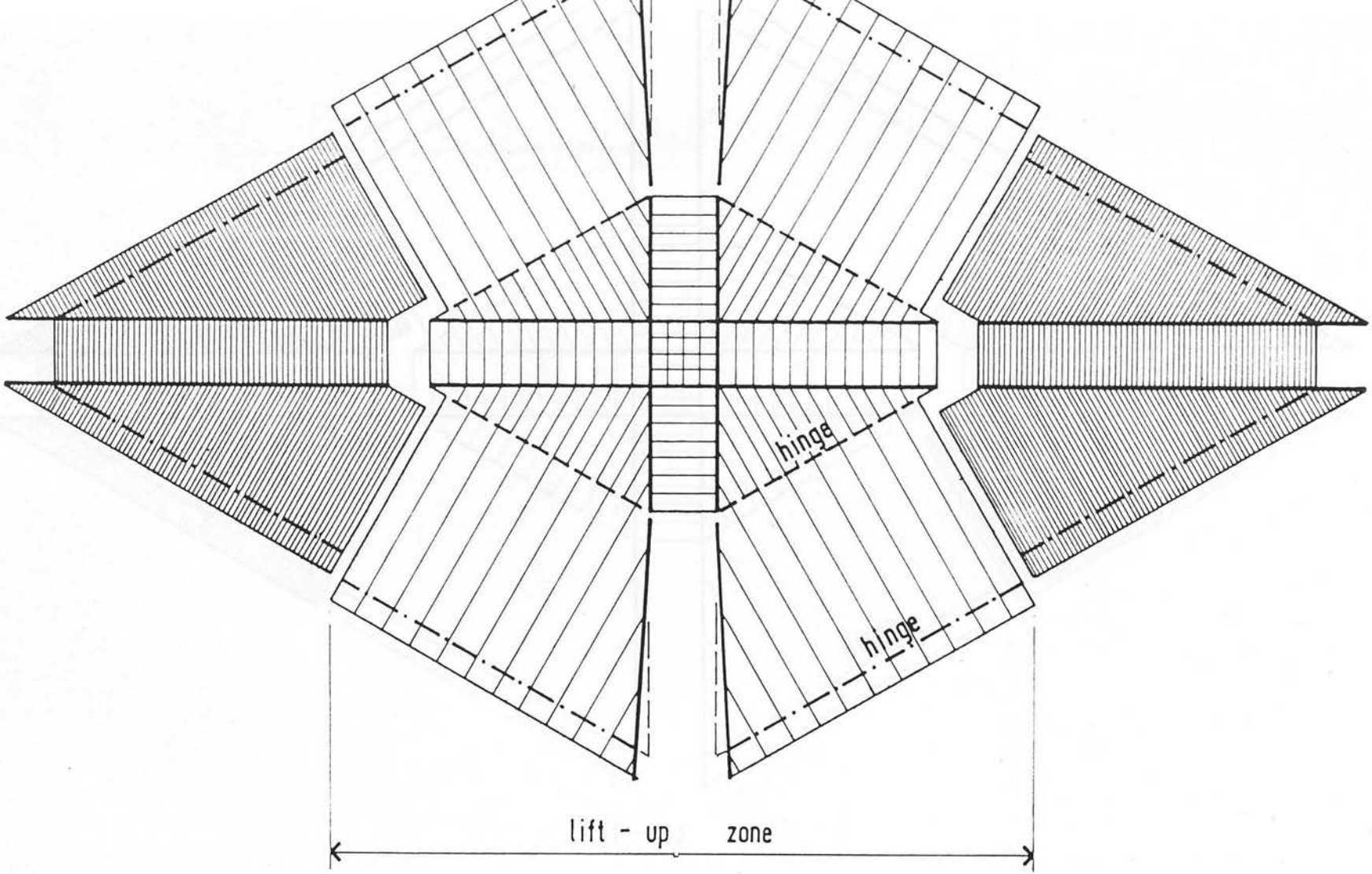
PANTADOME ERECTION SYSTEM



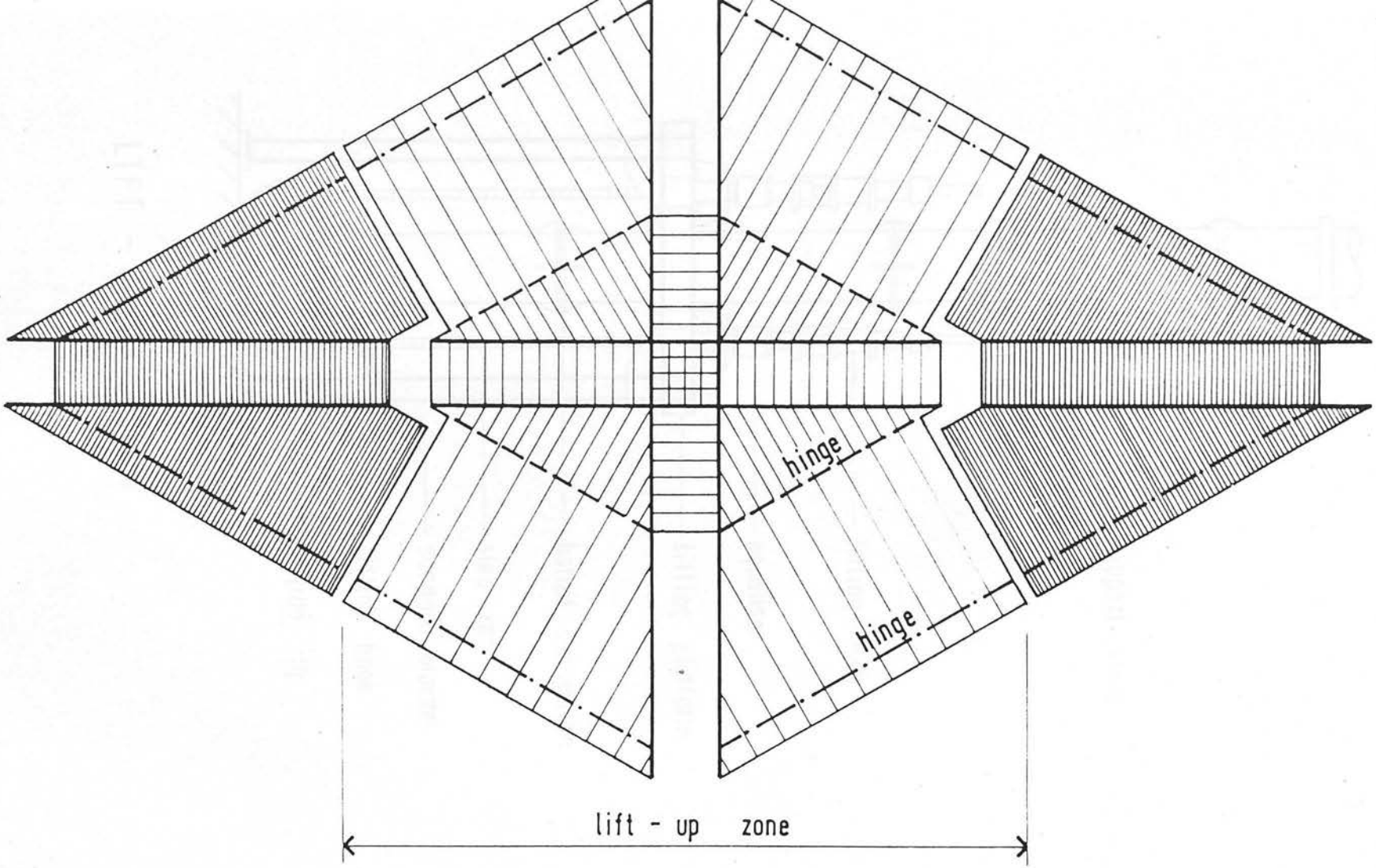
SPACE - FRAME : ROOF PLAN



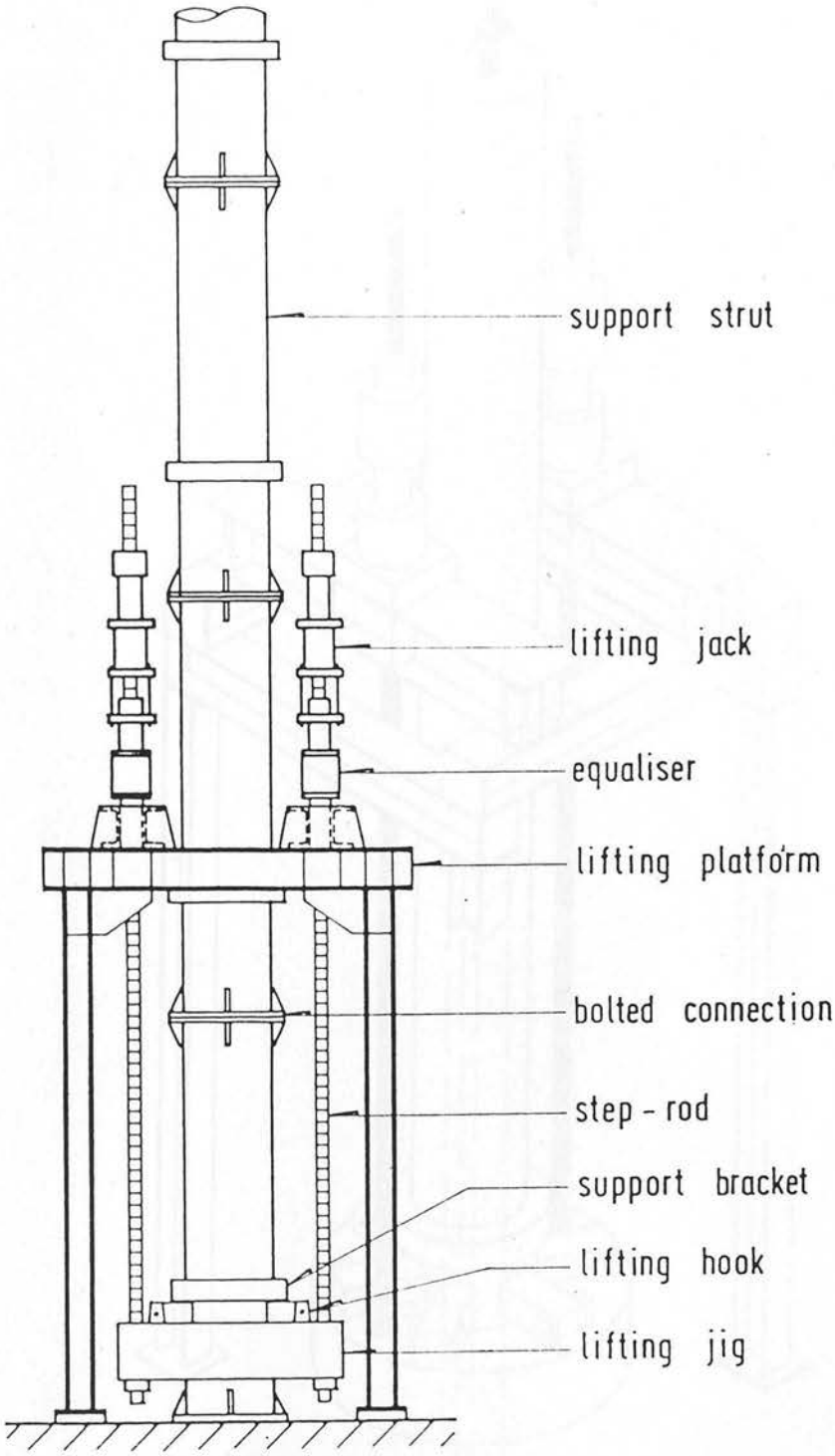
ROOF ERECTION AT LOW LEVEL



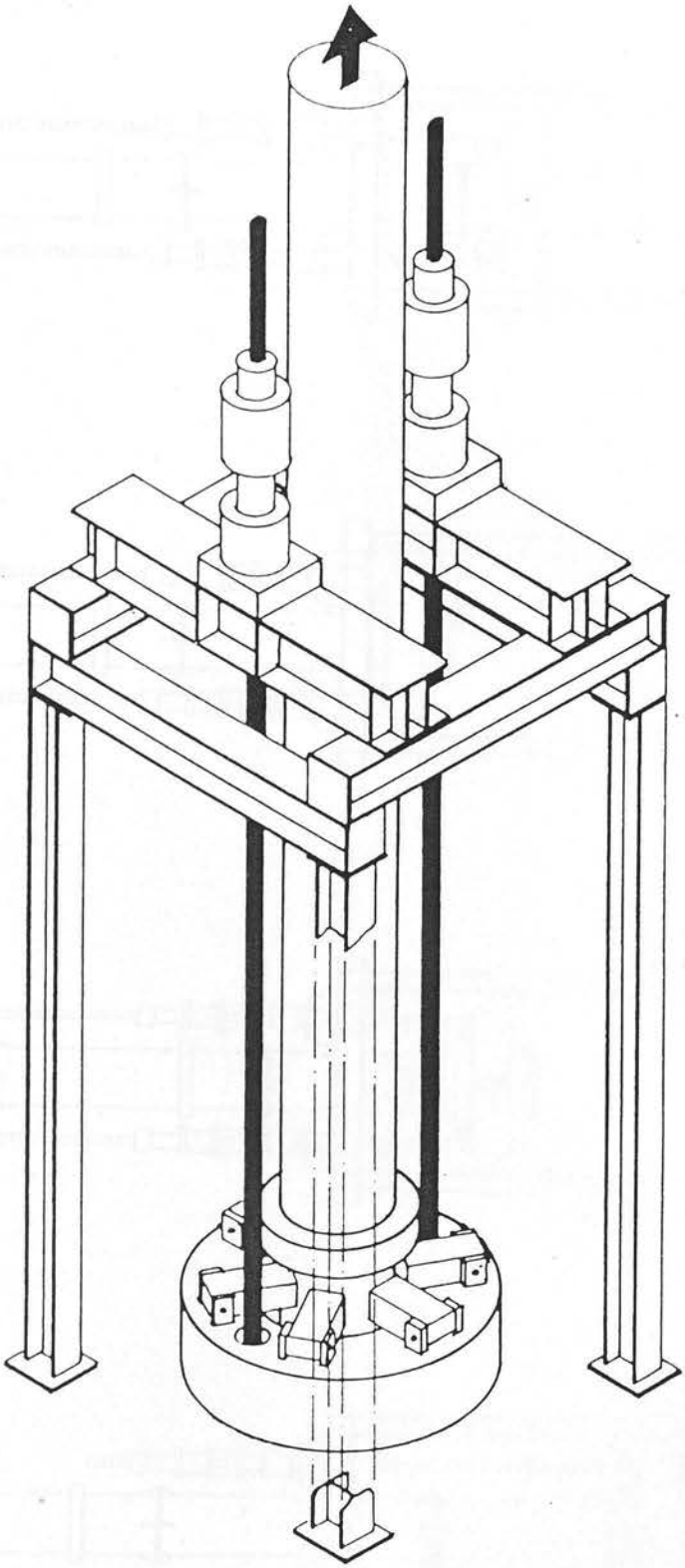
ROOF LIFT - UP



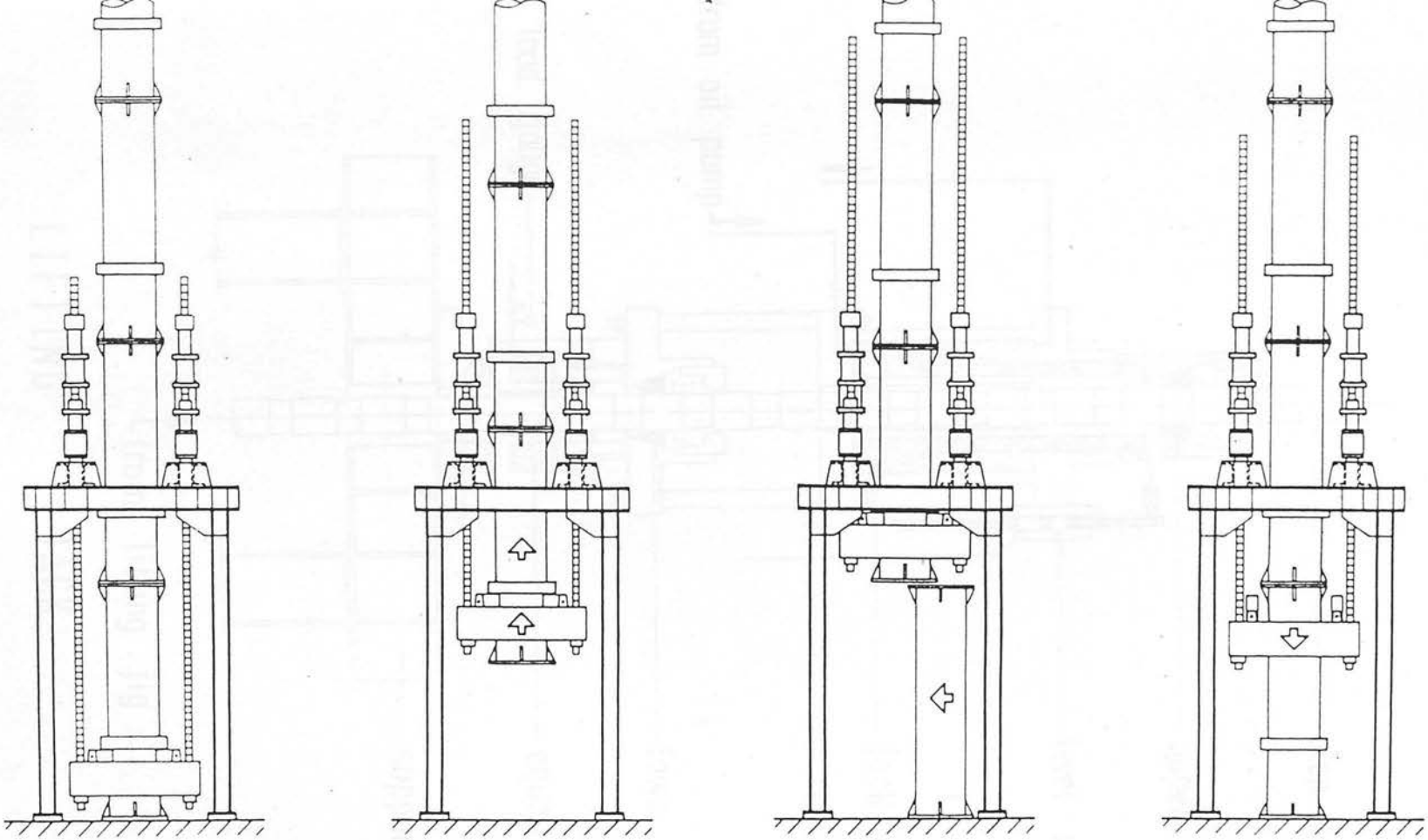
FINAL ROOF POSITION



LIFT - UP SYSTEM

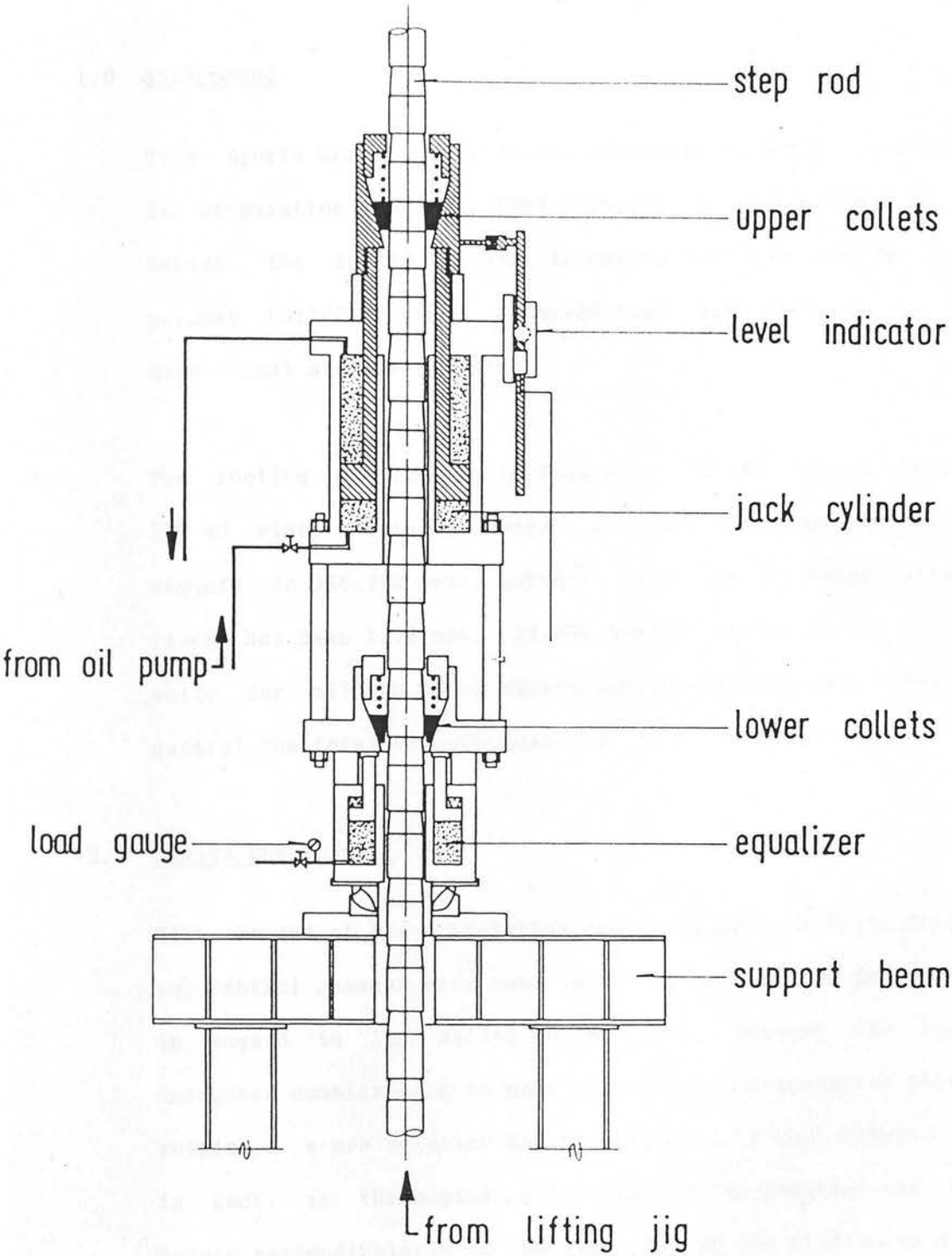


LIFT - UP SYSTEM
ISOMETRIC VIEW



STAGE 1 STAGE 2 STAGE 3 STAGE 4

LIFT - UP SEQUENCE



LIFTING JACK

CASE STUDY 44

(Extract from a Report on "Barcelona Sports Palace 1992 Olympics" by A Isozaki, I'arca, November 1987 Issue)

1.0 Background

This Sports Complex project was submitted in a 1983 competition in preparation for the 1992 Olympics in Barcelona. In this scheme, the Sports Palace is merged with the smaller multi-purpose building under a curved roof supported by a three-dimensional steel structure.

The roofing of the Sports Palace is 136.80 metres long and 110.40 wide, while the overall interior volume of the building amounts to 346.340 cubic metres. When the 200 metre athletics track has been laid out, 13,000 spectators can be accommodated, while for all the other sports activities and other events in general the total capacity can rise to 17,000 seats.

2.0 Design Development

With the end of the competition for the Olympic Ring at Montjuic, substantial changes were made in the design for the Sports Palace in regard to its siting in the area, because the team of designers commissioned to plan the general urbanisation there had submitted a new solution for the planning of the Olympic Ring. In fact, in the beginning the plan was to position the Sports Palace perpendicularly to the main axis of the stadium as a free-standing architectonic element. Later, in an advanced stage of the project, the site was changed, given the difficulty of eliminating the solid urban refuse deposited in the area, and the

planners exploited the differences in height produced by an old 30 metre deep sandstone pit. The solution then adopted was to shift the Palace towards the stadium, where the dump was less deep and involved a much smaller area. Because of this change the project was totally revised with respect to the foundations and the lower structure of the building, which were now in direct contact with the terrain.

Soon after planning the roofing of the Sports Palace, which developed the idea of a freely curved form, the engineers realised that owing to the enormous dimensions of a few points in the structure required where the contraction of the loads was greatest, the limits of resistance of the system of space games would be greatly exceeded. This initial "free-form" was later rationalised to the present form of the roofing, which has undoubtedly turned out to be not only more functional but also economical and feasible.

3.0 Architectural Concept

From a conceptual point of view the great roof of the Sports Palace was thought of as a series of curves that integrated the building with the terrain of Montjuic. The idea is reflected explicitly again in the lower part of the roofing, on a more intimate scale, in the form of a cantilever roof. The hierarchy between the different cladding elements (the main and the lower ones) is adapted from those of traditional Japanese temples pavilions (Garan), in order to create visual effects of nearness and distance between the different parts of the construction.

4.0 Erection of Structure

The interior height of the Sports Palace reaches 45 metres, and its space-frame roofing is used for anchoring both cladding elements and technical equipment. The roofing structure with all its finishings and equipment would be mounted on the ground and then raised by means of a system of hydraulic jacks to a height of 45 metres. After raising the structure and connecting the various sectors into which it was necessary to sub-divide the whole structure in order to raise it, the planners could then form a dome-shaped roofing supported by 62 perimeter columns. This system, called the Pantadome, derives from the pantograph mechanism in electric trains, widely used in Japan.

Flanking the Sports Palace is a smaller 'multi-purpose' building accommodating four basketball courts. The form of the building is that of a simple parallelepiped, with a roofing system, however, of a unique kind, consisting of girders whose geometry clearly reflects the transmission of stress and the typical compressions of the structure.

The Pantadome system is not dissimilar to the one used in the Festival Plaza roof structure in Expo 1970. Its innovation lies in the use of a spherical ball-joint to resolve the problems of space-frame construction:

Kawaguchi[1] summarises the problems as those which stem from the gaps between demands in measurement and angle accuracy, and the

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(1) Kawaguchi, M "The Structure of the Space Frame Roof Over Festival Plaza" p 20, J A, May 1969.

limits imposed by on-site operations ie the design tolerance of one millimetre is not always realistic, even by normal standards of accuracy on site, with the result of misalignment being a common problem to space frame assembly.

The answer to this lay in the development of a jointing system which allowed both variations in angular alignment and in length caused by inaccuracies during assembly.

The ball-joints were cast with access openings to allow the insertion of bolts from the inside. The heads of the bolts are fitted into a sphere concentric with the main sphere of the ball joint. A clearance of 6 mm between the sides of the access openings which provided the bolt axis with a tolerance in radian tension resulting from angular variation from the joint centre.

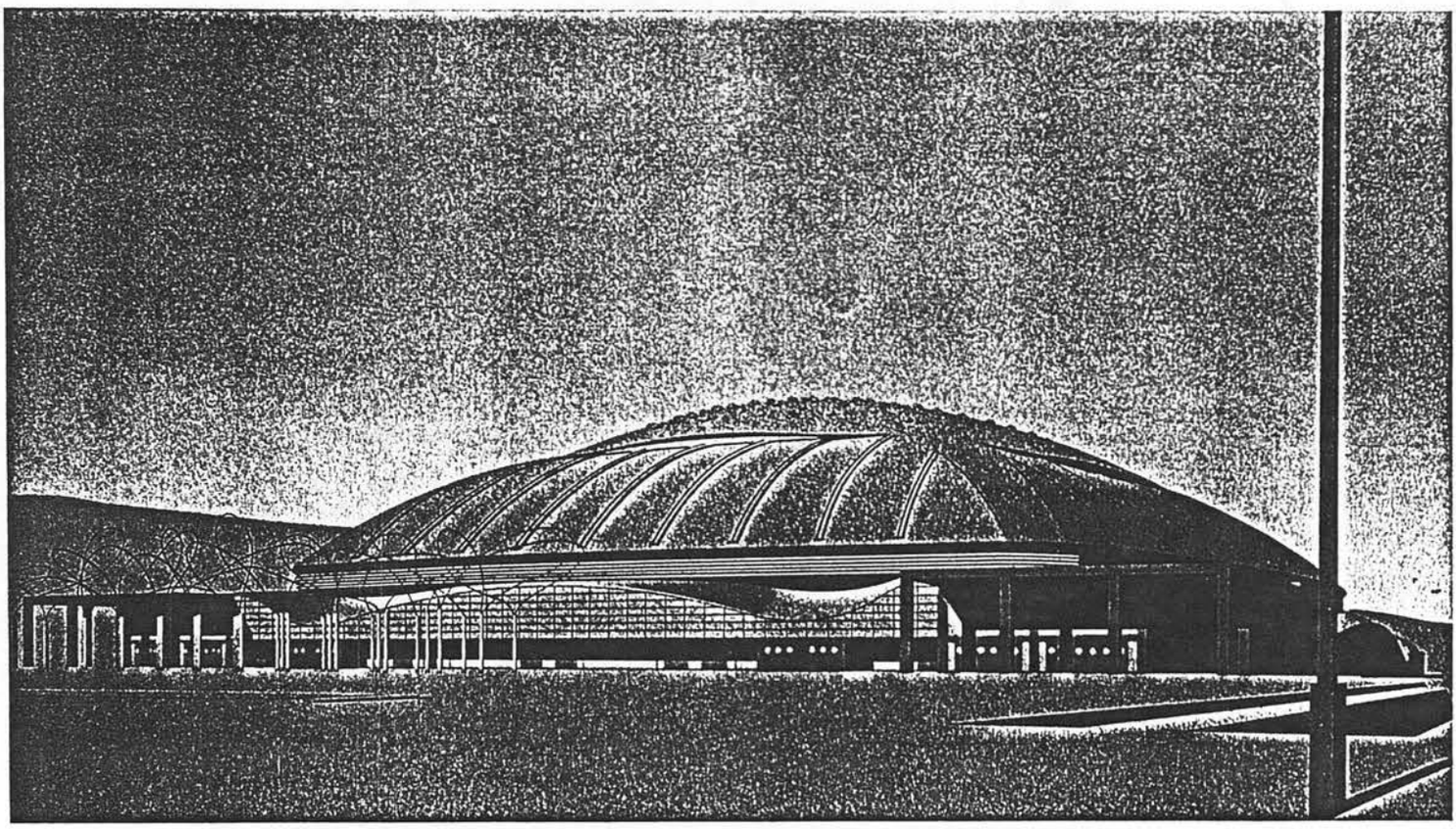
The shaft of the bolt bore two sets of threads for connection with the chord members during on-site erection. One set was used to absorb the error by absorbing tension and the other to resist compression. The discrepancies in both the lengths and angles of member connections were thus limited to less than +/- 0.5 mm. The bolts could be cast from chrome molybdenum steel or nickle-chrome molybdenum steel.

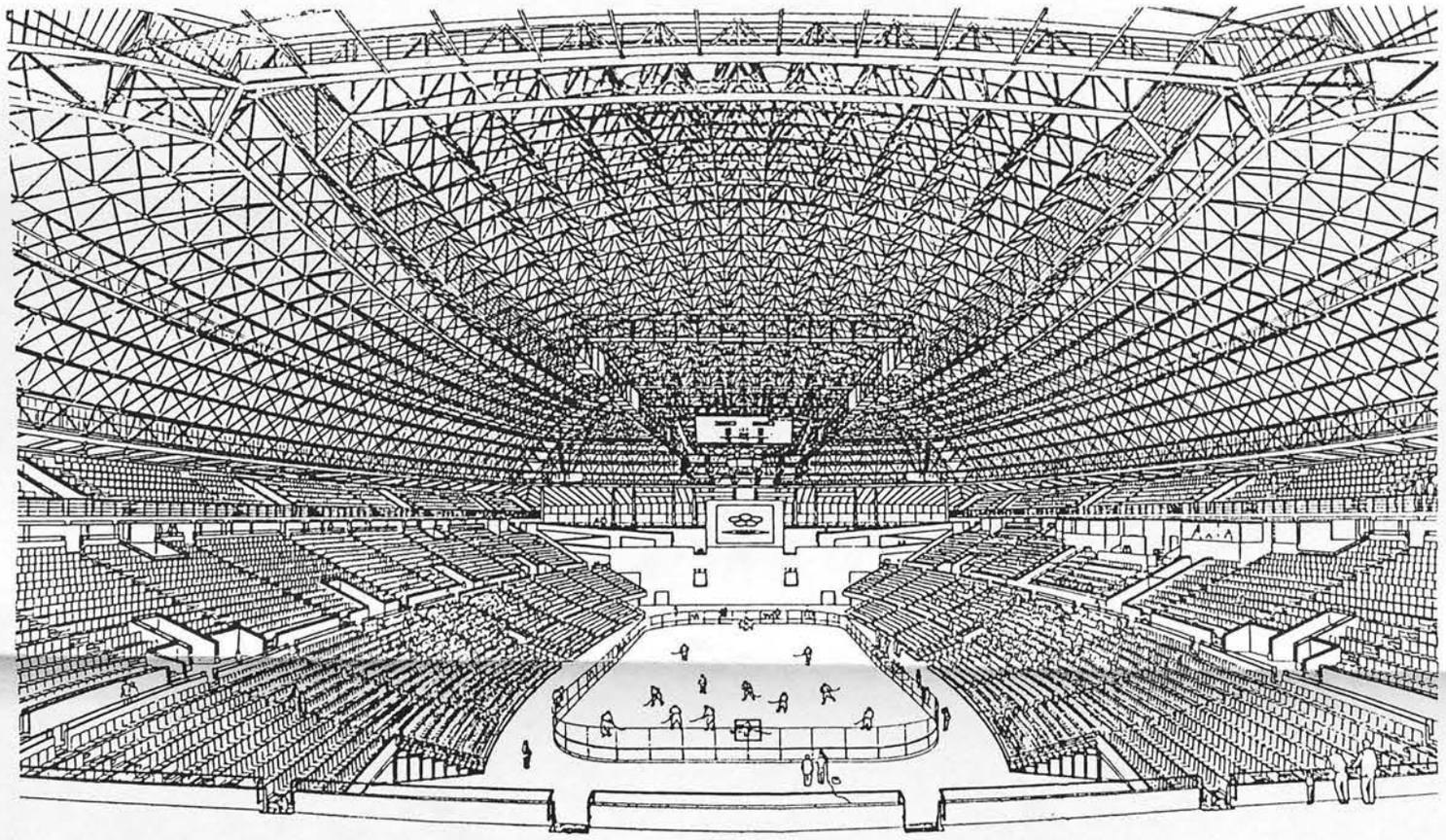
■ *Prospettiva della facciata principale del Palazzo dello Sport di Barcellona.*

■ *Perspective of the main façade of the Sports Palace of Barcelona.*

■ *Vista prospettica dell'interno del Palazzo dello Sport, caratterizzato dalla grande copertura metallica a maglia spaziale.*

■ *Perspective view of the interior of the Sports Palace characterized by the spatial grid structure of the metal roofing.*

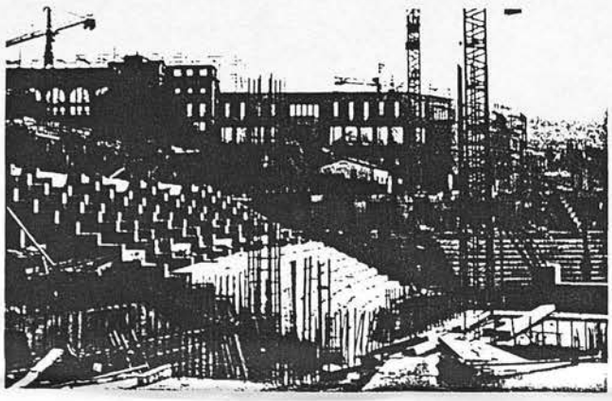


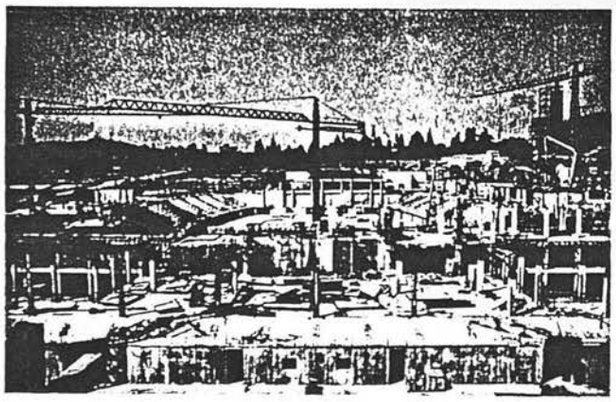


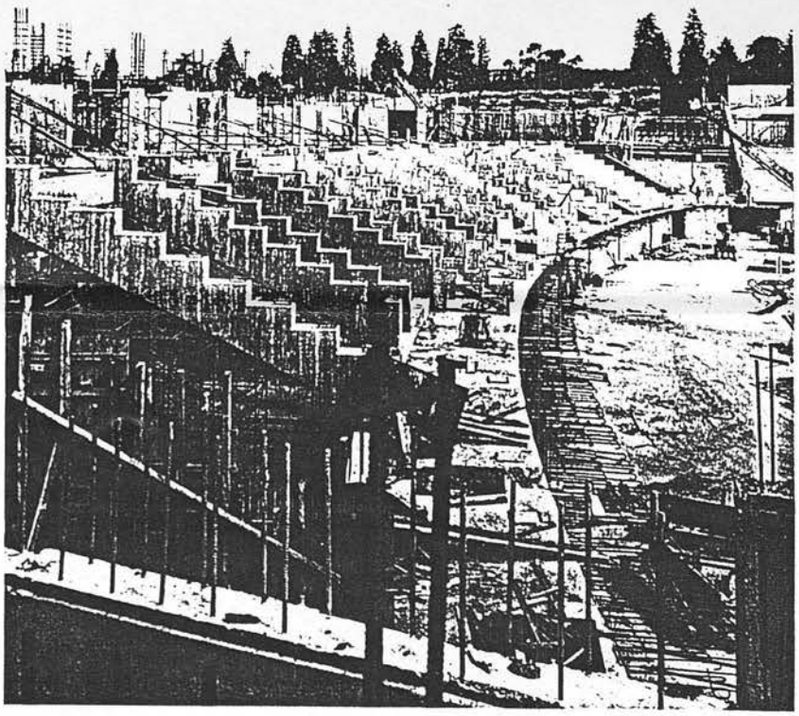
■ *Vista del cantiere del Palazzo dello Sport con lo stadio di atletica in costruzione sullo sfondo.*
■ *View of the site of the Sports Palace with the athletics stadium under construction in the background.*

■ *Vista generale del cantiere del Palazzo dello Sport.*
■ *General view of the construction site of the Sports Palace.*

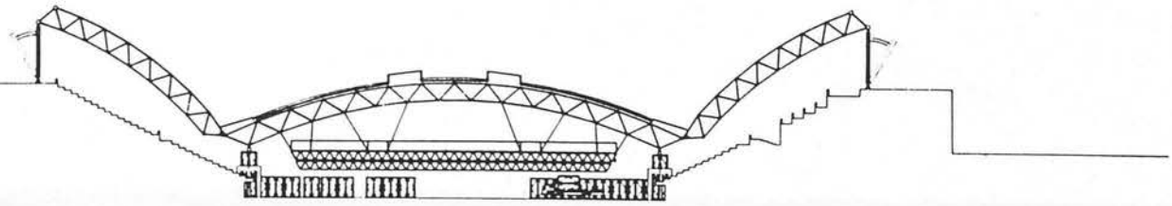
■ *Le strutture delle tribune in fase di costruzione.*
■ *The stand structure under construction.*



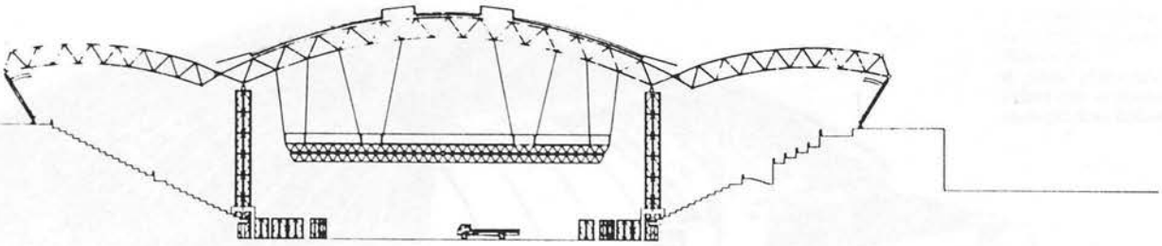




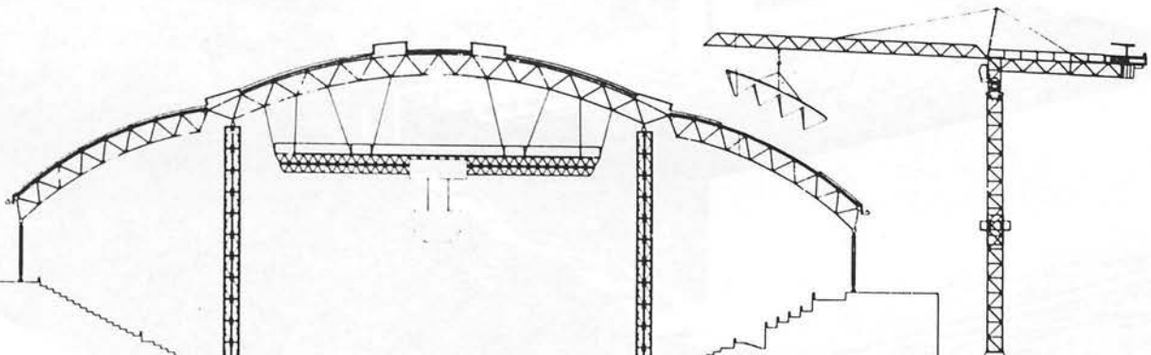
■ *Sistema di elevazione "Pantadome" della struttura di copertura: prima fase di montaggio.*
■ *The "Pantadome" system of raising the roof structure: initial assembly phase.*

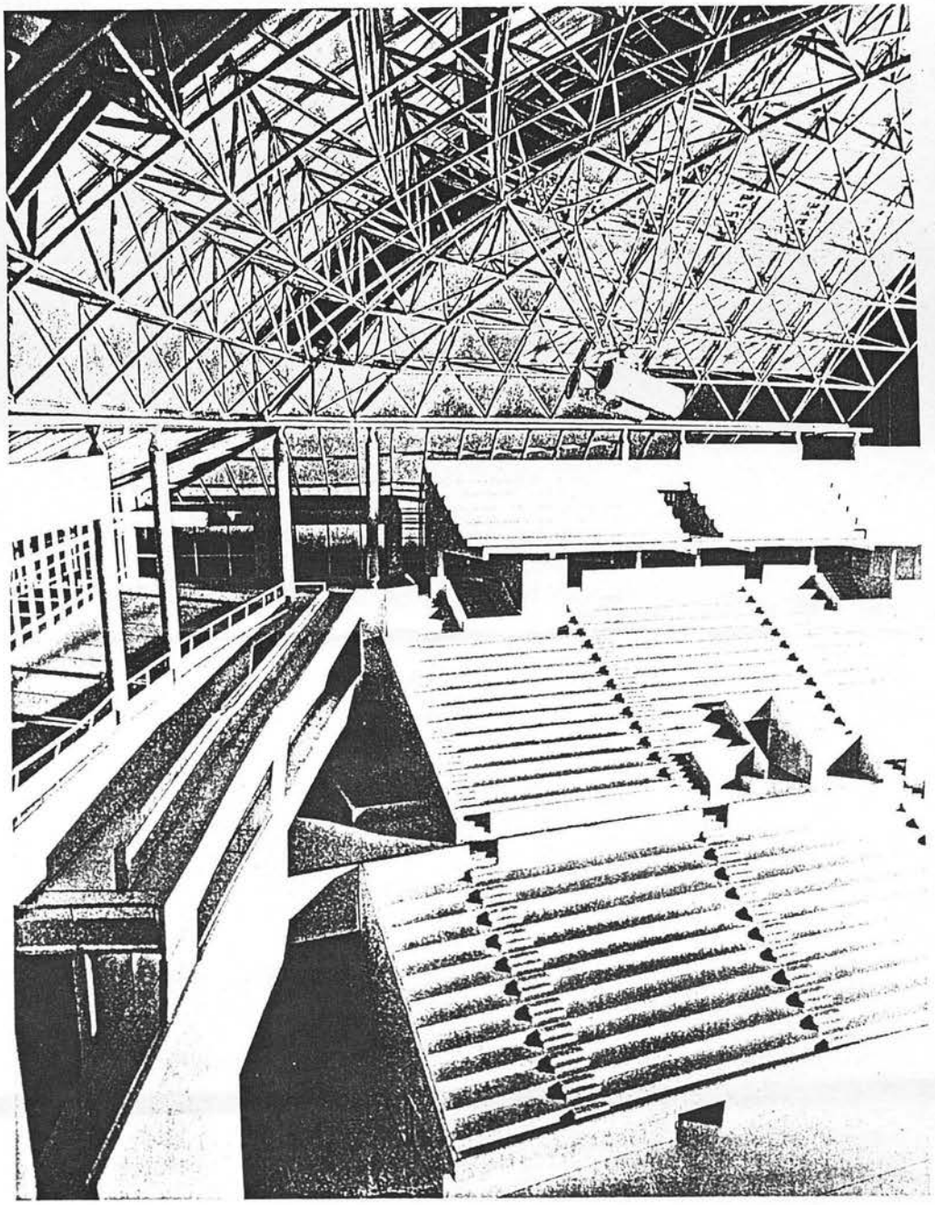


■ *Fase intermedia di innalzamento delle strutture.*
■ *Intermediate stage of the structural elevation procedure.*



■ *Fase finale del montaggio della copertura metallica.*
■ *Final phase of the assembly of the metal roofing.*





■ *Vista del modello dall'interno.*
■ *View of the interior of the model.*

Credits
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Security, plumbing and drainage:
Francisco Labastida
INYPESA
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Santiago Sarrate
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Toshio Ojima
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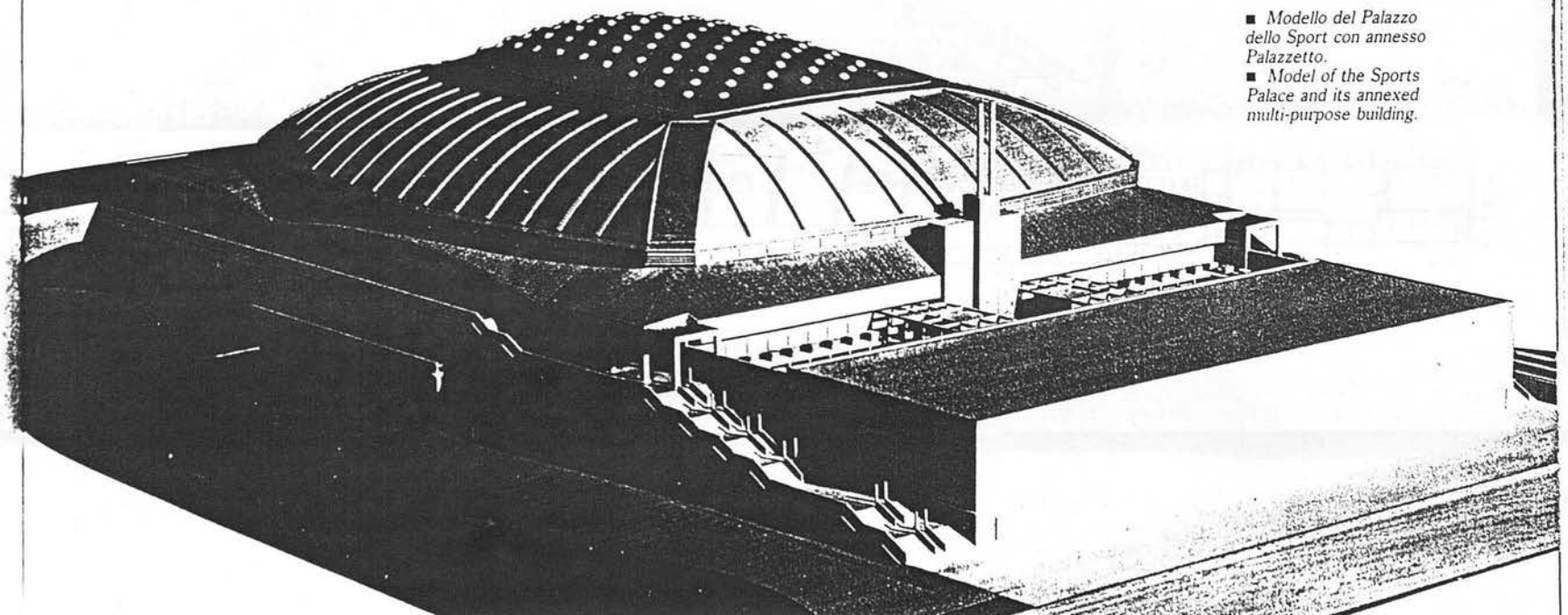
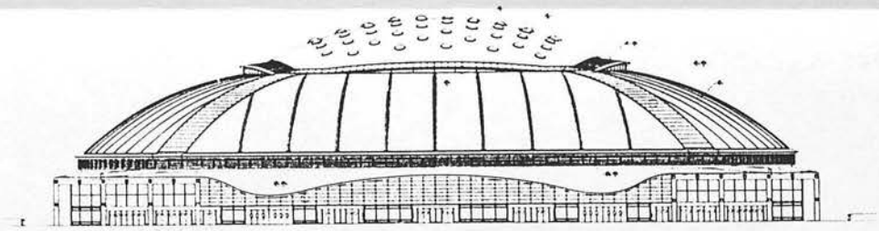
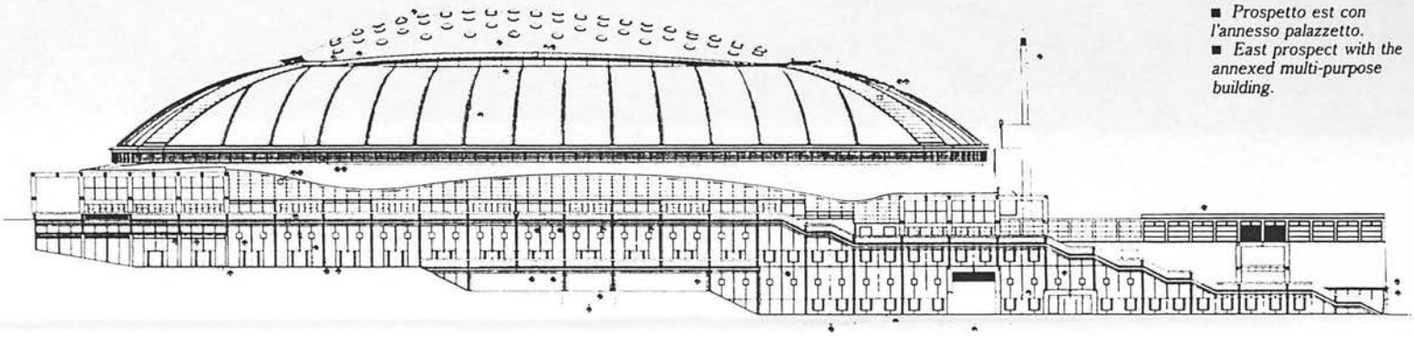
Shuzo Murakami
Acoustics:
Hideki Tachibana
Drawings:
Lluis Punter
Herminia Cordoba
Models:
Juan Pablo Marìn
David Correa Lòpez



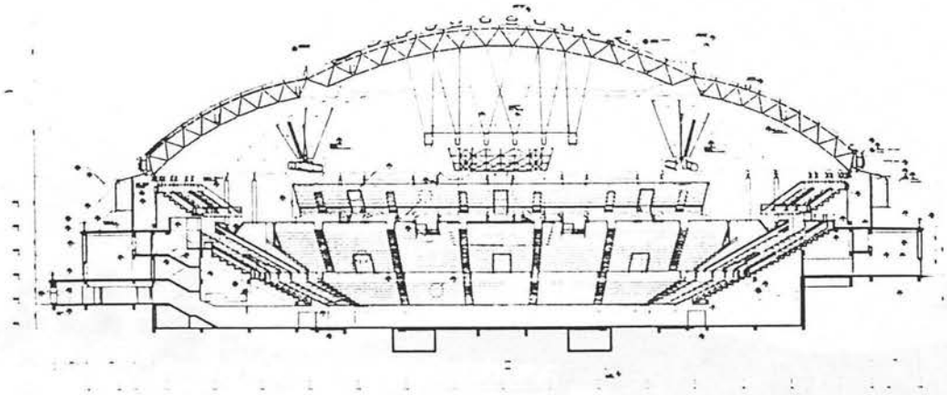
■ *Modello del Palazzo dello Sport con annesso Palazzetto.*
■ *Model of the Sports Palace and its annexed multi-purpose building.*



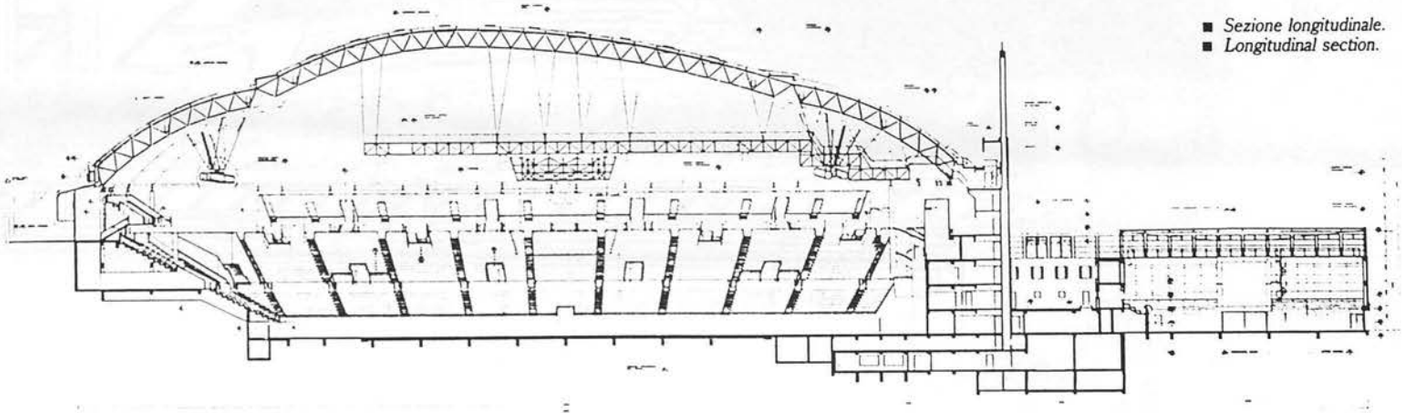
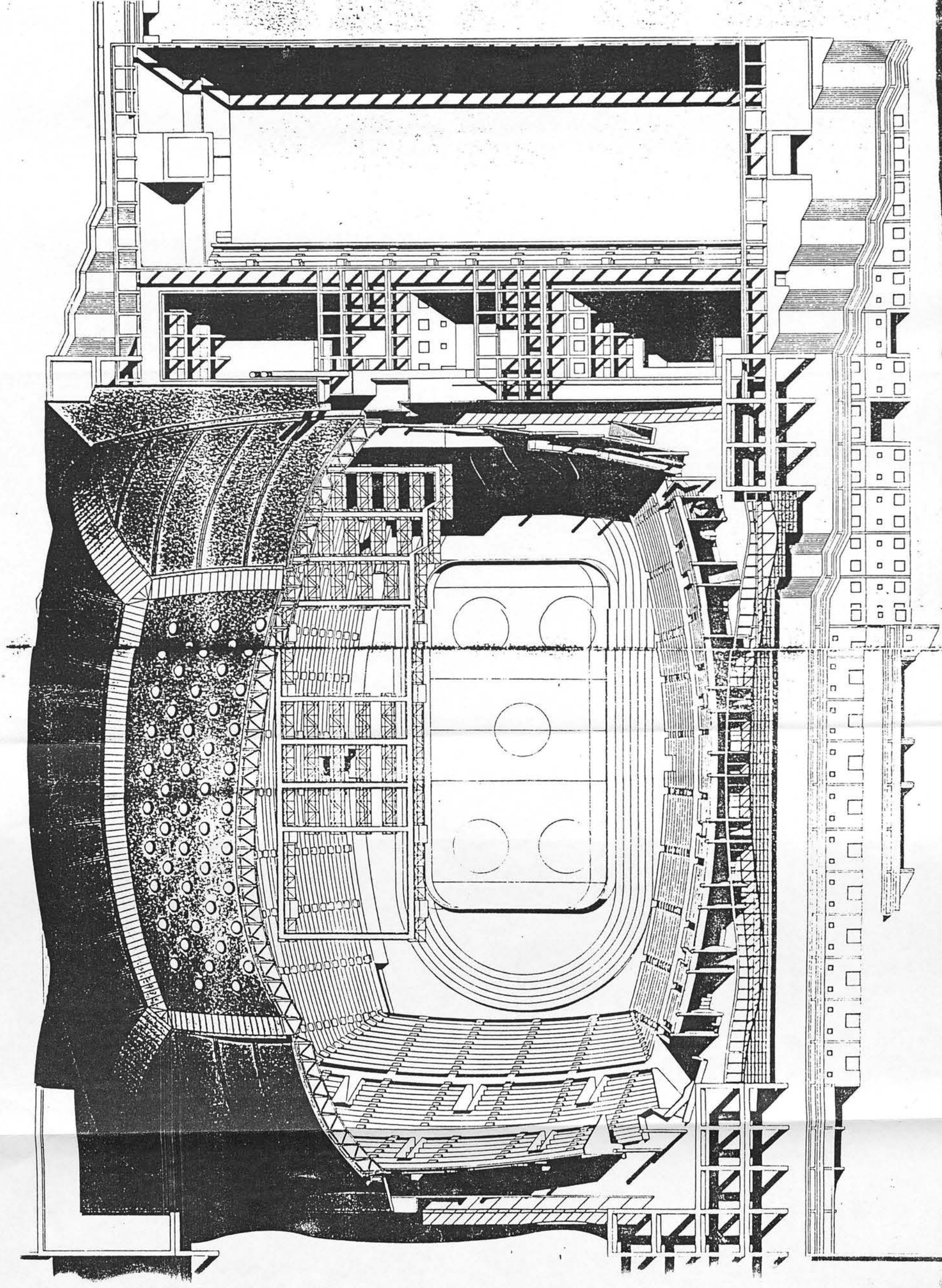
■ *Prospetto nord del Palazzo dello Sport.*
■ *North prospect of the Sports Palace.*



■ *Prospetto est con l'annesso palazzetto.*
■ *East prospect with the annexed multi-purpose building.*



■ *Sezione trasversale.*
■ *Cross-section.*



■ *Sezione longitudinale.*
■ *Longitudinal section.*