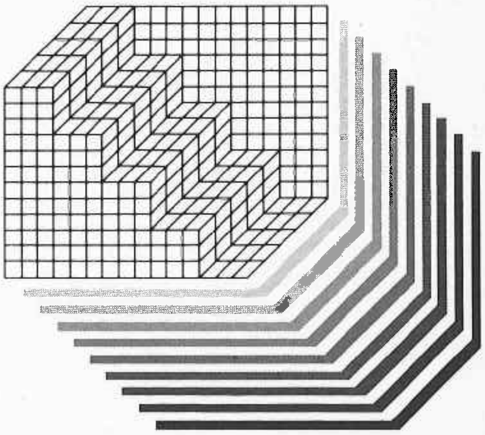
Patterns 8



Patterns 8

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Contents

Apologies The editors apologise for the unintended omission of credit to Holder Mathias Alcock as architects on York House in Patterns 7.

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A Class of schools – a comparative study of West Totton, Bishopstoke and Fleet Velmead Schools, Hampshire

Project data

Client	Hampshire County Council
Architect	Hampshire County Council
Structural Engineers	Buro Happold
Services Engineers	G H Buckle and Partners
Quantity Surveyors	Gleeds/HCC
Main Contractor	T A Brennan Ltd
Glulam Subcontractor	Sherry and Haycock
Project Value	£1m
Completion Date	September 1990

Seldom do engineers have the chance to design to identical briefs, on nearly similar sites, but as part of different design teams while working for the same client. The infant schools at Fleet Velmead (1986, Architects Michael Hopkins & Partners), Bishopstoke (1988, Architects Hampshire County Council) and West Totton (1989, Architects Hampshire County Council) are such an occurrence and it is useful to compare the design solutions of each.

Further, it is to be hoped that these three short articles illustrate the benefits gained by a public body of architects, such as Hampshire County Council, expending the diversity of their design interests into a wide range of buildings. Every aspect of the County Education Department's requirements as defined by the Department of Education and Science's guide notes, in particular Design Note No 17, is co-ordinated in the articulation of all the physical aspects of building construction including lighting, ventilation and structure into the finished architecture.

Being infant schools, operating mainly by day, each school is driven by its need through its section to introduce natural light deep into the classroom plan, using central circulation as shared 'storage' and 'resource' areas. All three schools utilise this possibility in interesting but different ways in the organisation of both plan and section.

Some consideration of the differences between the three schools is interesting, since each is heavily influenced by choice of engineering solution. There could not be two more different approaches to the structuring and servicing of a nine class infant school than between that adopted at Fleet Velmead and Bishopstoke. Both are outwards facing to exploit their surroundings but in very different ways. While the former is a low key design of modern tubular steel on a rectangular grid, the latter deliberately has its roots in the craft of building, is high profile and idiosyncratic in its radial spiral plan, and is demonstrative in its structural details. On the other hand, West Totton is deliberately low key, friendly, and constructed purposely of low energy building materials using timber, brick and simple reinforced concrete construction where possible.

It also seems opportune to write by way of a kind of tribute, about some of our work for Hampshire County Council in the year in which their Chief Architect, Colin Stansfield-Smith, was awarded the RIBA Gold Medal.

Michael Dickson

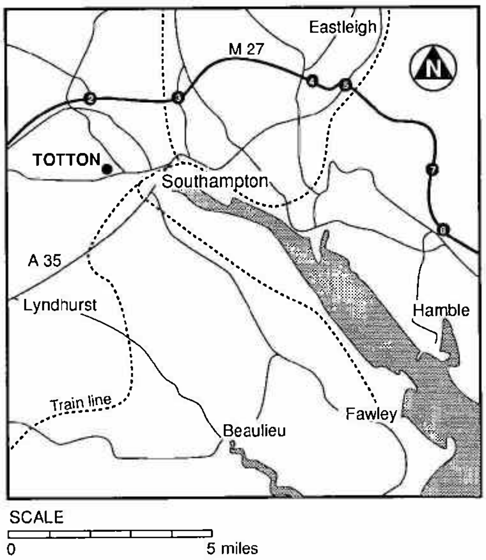


Fig 1.1 Location of West Totton, Hants.

West Totton First School

The six class West Totton building is on a small flat site, entirely surrounded by housing and in part overflown by a national grid power line (Fig 1.1). The building looks inwards with the classrooms in the side blocks of the gridded H plan while entrance is from the south where the higher hall, music and library block separate the two classroom blocks (Fig 1.2). The roof to the classrooms tilts upwards from the mainly solid side brick walls to open the classrooms visually through full height glazing into the northern courtyard (Fig 1.3 a, b). The clerestory side glazing together with the end glazing to the central hall and library block allows good natural light to penetrate longitudinally into the occupied spaces (Fig 1.4).

The exposed, planed timber grid roof construction exploits the possibility in modern laminated timber of economically using 'small' pieces of wood accurately prefabricated in order to build up a stronger, stiffer composite structure – one which in its completed state is easily understood by the user. At the same time the roof's structure provides a visually pleasing but organised construction system from renewable materials.

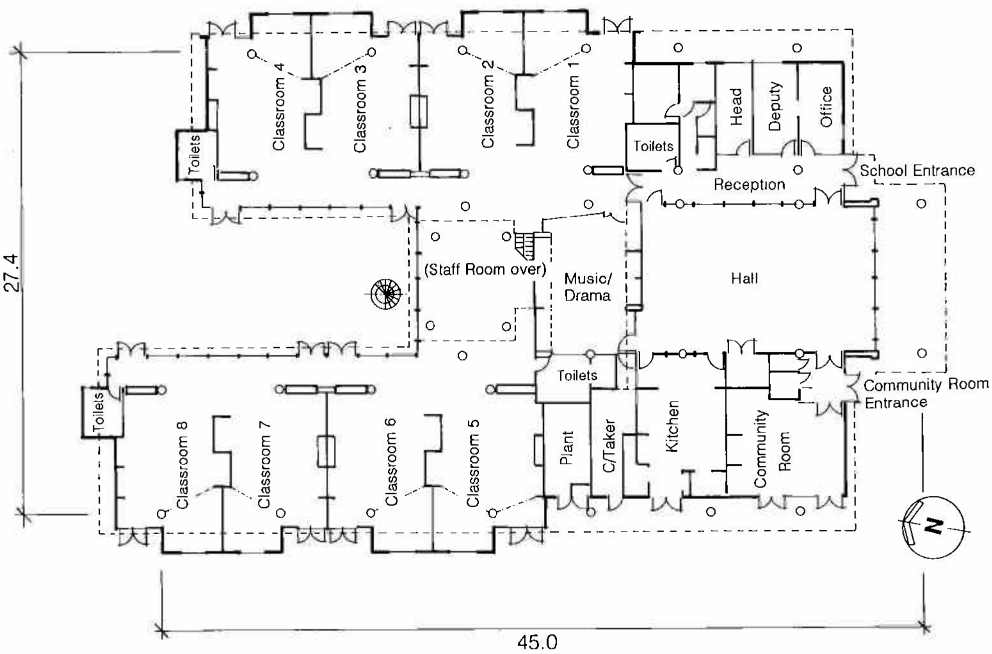


Fig 1.2 Plan of classroom and administration layout

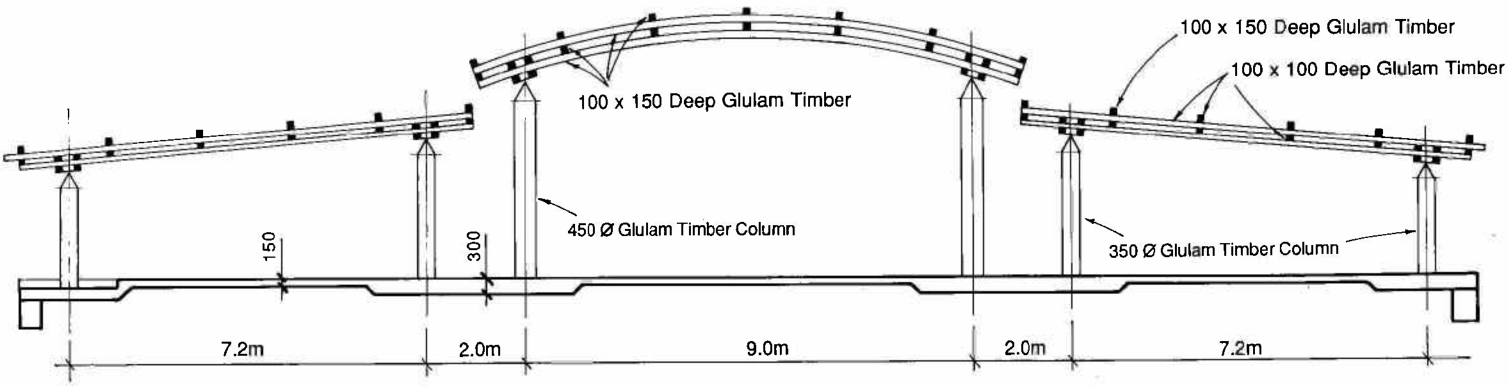
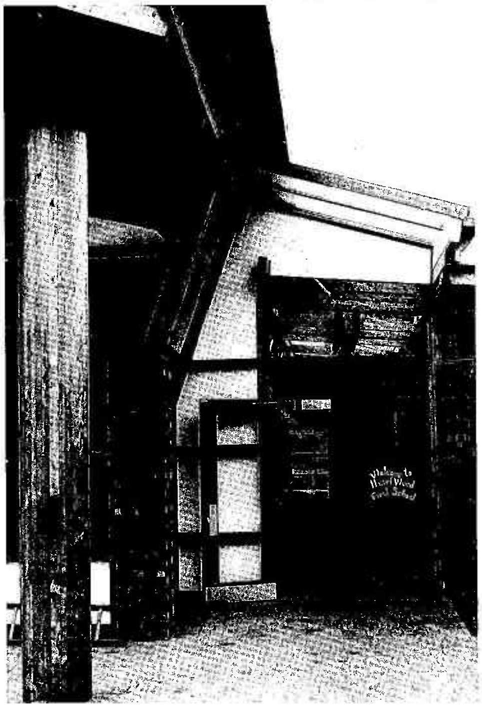


Fig 1.3 a) Section through roof and side blocks of school
b) 'Pencil' like column supporting sloping roof



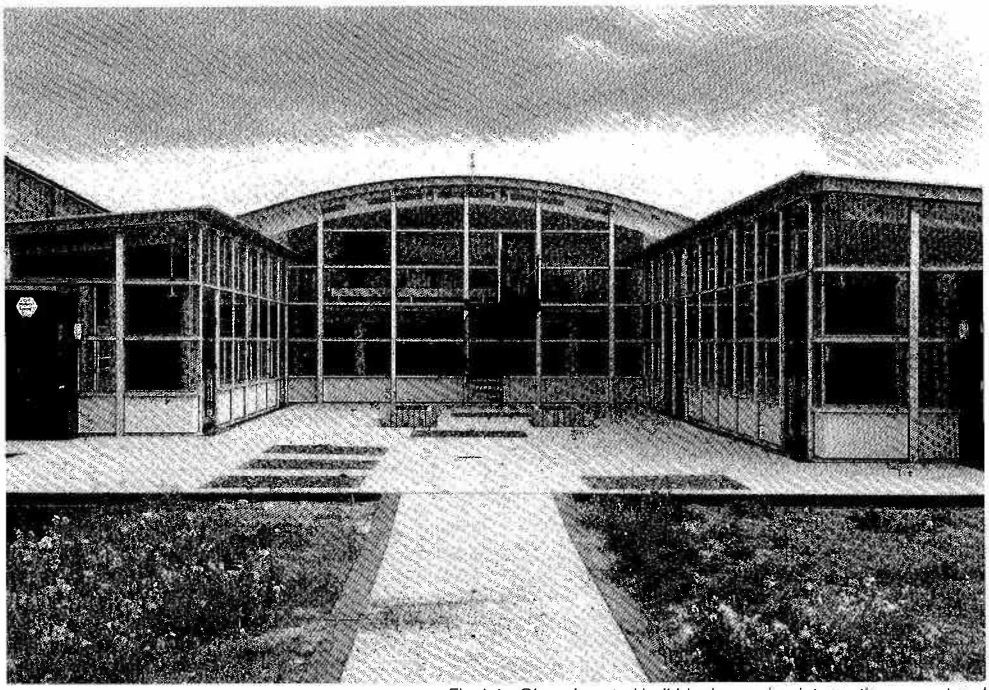


Fig 1.4 Glazed central hall block opening into northern courtyard

Glued laminated timber

Mechanically-laminated timber structures date back to the 1800s, the best known examples being the railway arches designed by Brunel. However, it was not until the 1940s following the development of powerful, synthetic resin adhesives, that glued laminated timber (glulam) as we know it today began to emerge.

Because of the variability of strength properties in timber, working stresses for sawn timber are considerably lower than the mean values to allow for the weaker components. If two pieces of timber are glued together and subjected to the same strain, the stress in each individual piece depends upon the difference in the moduli of elasticity. Now the variability works in reverse, and the higher strength of one piece compensates for the lower strength of the other. This results in significant increases in the allowable working stresses and stiffness, and hence the achievement of lighter and more efficient structures.

Roof design

Glulam can be prefabricated to a large extent and this assisted in the development of a scheme at West Totton where design and method of construction were very closely related.

The roofs are formed from a lattice of timber glulams. In the classrooms the sloping space grid roof is a double layer of 100mm × 100mm laminated timbers on 1.8m module, on spun circular laminated timber 'pencil' columns (Fig 1.5 a, b). Over the 9m wide hall, music room and library the cylindrical laminated timber grid and window mullions use 150mm × 100mm laminated timbers.

Glulam ladder beams between 300 and 450mm overall depth and at 1.80m centres spanning in two

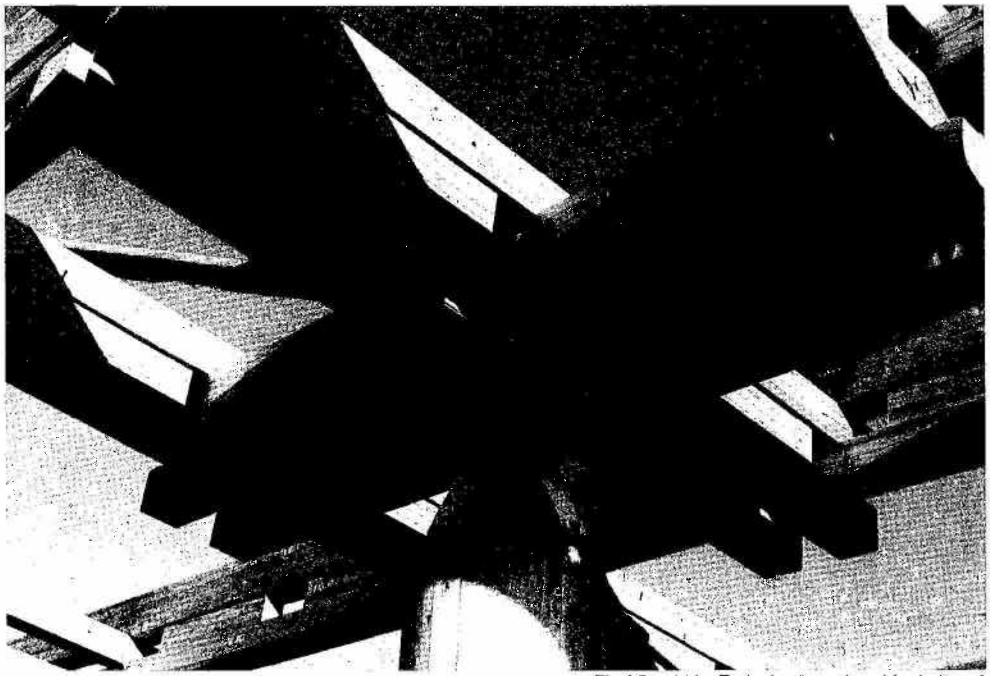
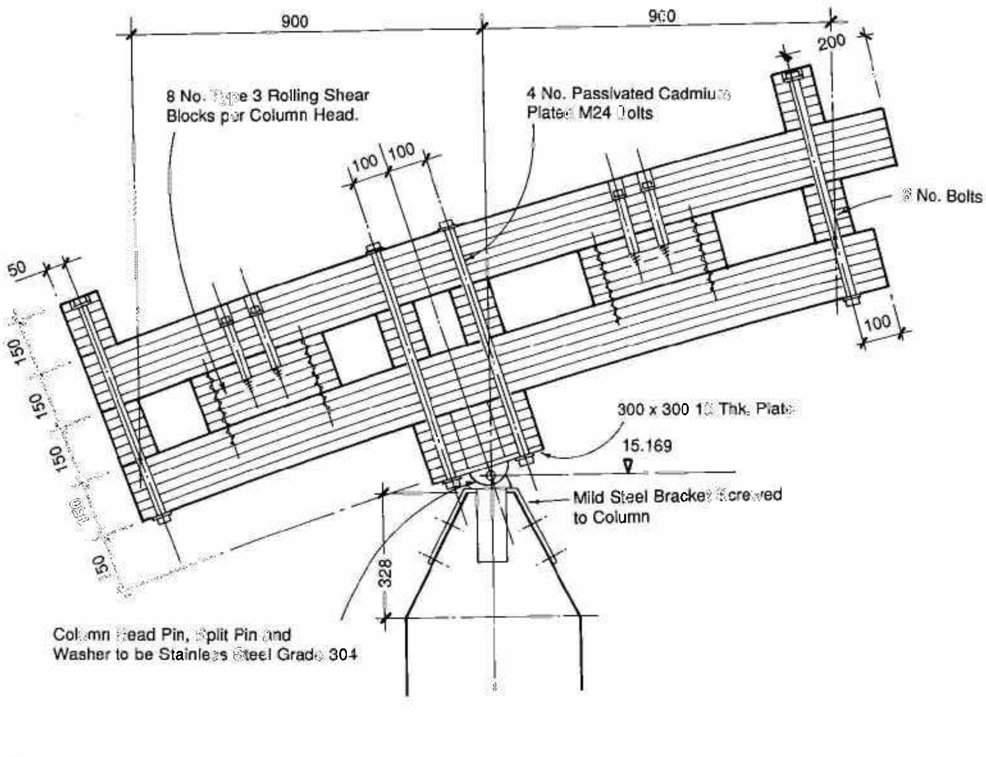


Fig 1.5 a) b) Typical column head for hall roof



directions forming a grillage are the structural basis for the roof. Where members cross, they are bolted together with a long M20 bolt for location purposes. The top and bottom member of each ladder beam (Fig 1.6 a) are connected with glulam blocks which transfer the horizontal shear forces, hence making the two members work as one beam. Blocks are factory glued to the bottom member and site joined to the top member. This site connection is made by tight-fitting mild steel sleeves, drawn into the joint with coach screws to provide the horizontal shear capacity, so ensuring a strong, stiff joint. To carry the punching shear at roof supports, a column head of larger glulam sections is used (Fig 1.6 b).

It was decided that shear blocks should be as small as possible to give the ladder beams a visually open and light appearance. Joints were therefore grouped to use three different types of shear blocks designed with an increasing length and number of screws, to transfer an increasing horizontal shear (Fig 1.7). The framework analysis of the roof established the required horizontal shear transfer throughout the roof, and the type and number of shear blocks could then be determined.

The relatively complex layout of shear blocks that resulted was considered acceptable as blocks could be glued in place in the factory from fabrication drawings produced by the manufacturer, and checked by the architect and engineer. In principle, the site work consisted of bolting together a kit of parts.

Stability of the building

The roof is supported by circular glulam columns, cone-shaped at the top and with brightly painted steel head detail giving each a pencil-like appearance. As there is no cross bracing between these columns, the stability of the building relies upon the fixity of the columns into the concrete slab. A base detail was designed with four mild steel bars epoxy-resined into the column and grouted into the slab (Fig 1.8), the thickness of which was locally increased under the columns to accommodate the bending moments.

Initially fixed bases were designed using modular ratio calculations and an assumed bond between the steel epoxy and timber. Although pull out tests had been carried out previously in Denmark, it was felt that these provided little reliable information considering the structural importance of the column base detail. It was therefore decided that as part of the contract, the manufacturer should supply five test pieces and from these prove that the calculated capacity of the fixed bases could be achieved. The results of the tests, carried out for the contractor by the Timber Research and Development Association (TRADA), showed little variability and were in accordance with both the Danish findings and the

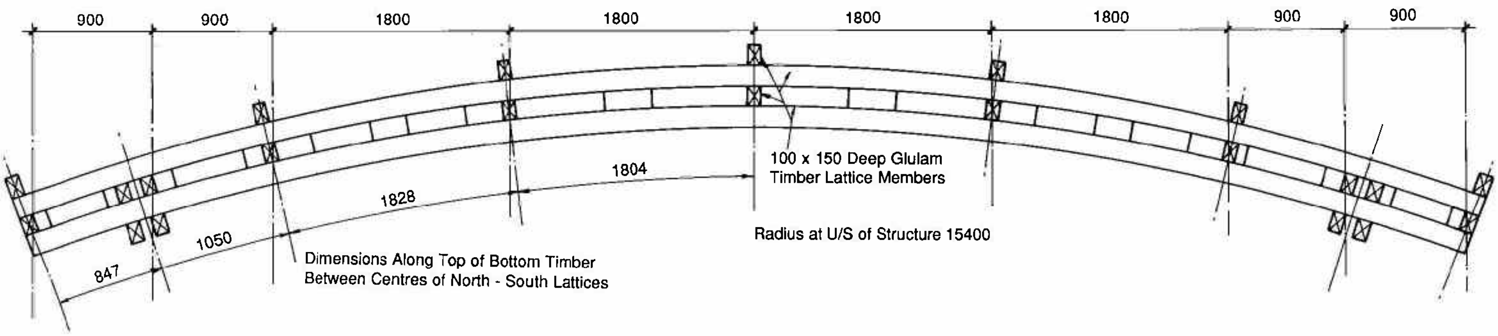
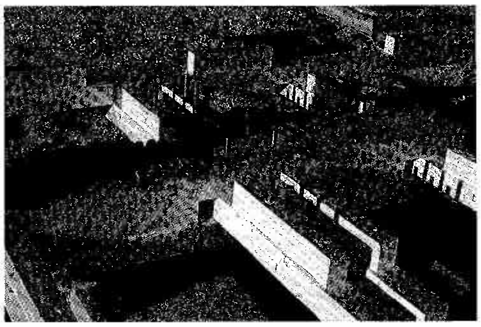


Fig 1.6 a) Transverse section through hall roof lattice
b) Assembly of column head



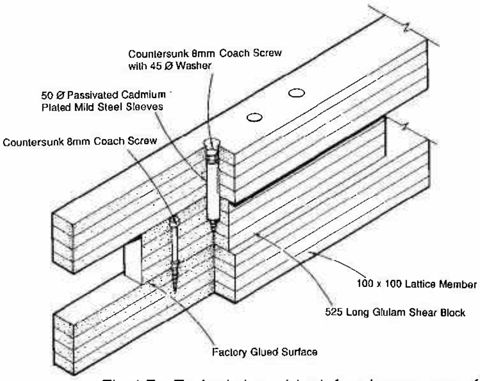


Fig 1.7 Typical shear block for classroom roof

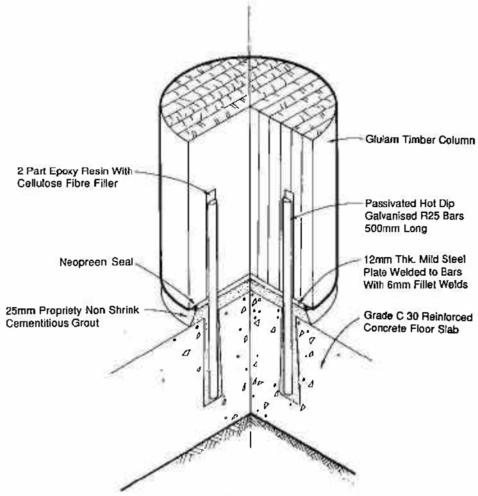


Fig 1.8 Typical column base detail

engineer's calculations. This gave sufficient confidence in the performance of the base detail.

Connection detail

Glulam was used consistently throughout the building, not only for the structural frame, but for elements including straight and cranked window mullions, mezzanine floor beams and a staircase. Consequently a large number of connection details were necessary. Where these were out of sight, simple angle brackets and framing anchors were used but visible connections were generally made using a concealed steel plate and bolts.

Erection of the frame

Sherry and Haycock were the successful tenderer for the glulam subcontract, and manufacture of the glued laminated timber was carried out for them by Limtrae Lilleheden in Denmark.

The initial design required the assembly of 10m × 15m roof sections on the ground slab. Each roof section could be given a permanent precamber by drilling holes for the sleeves in the shear blocks on site after the roof had been set up. This precamber was provided not for elastic deflection under dead load, but mainly to compensate for any slack in the coach screw/sleeve connections. The amount of precamber was largely based on engineering judgement of the stiffness of the shear blocks.

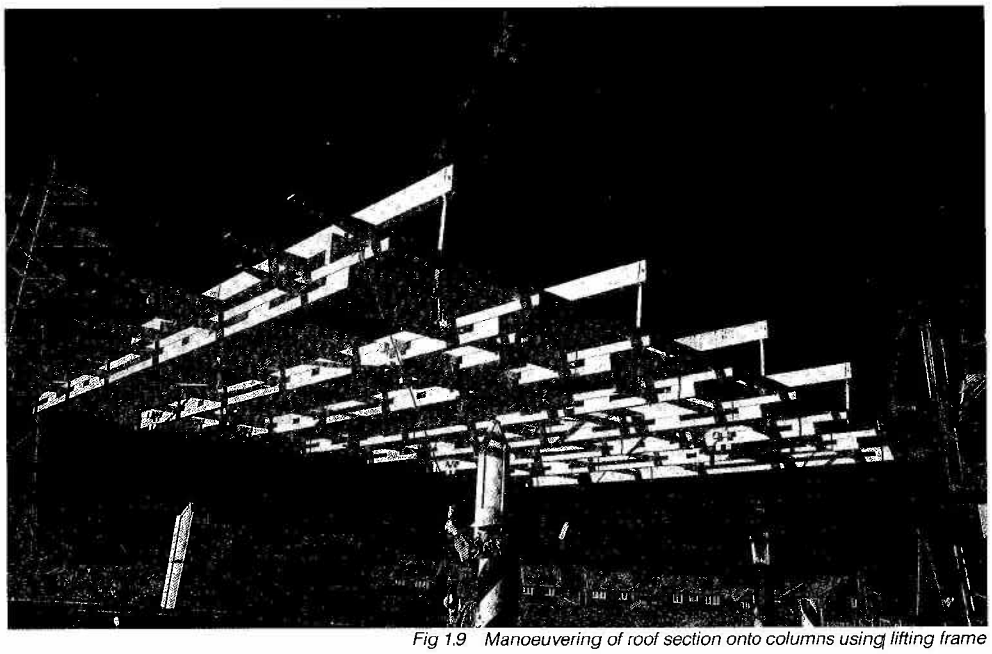


Fig 1.9 Manoeuvering of roof section onto columns using lifting frame

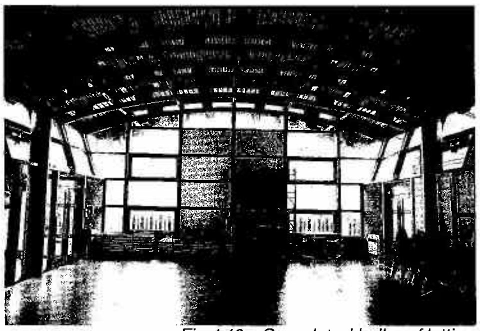


Fig 1.10 Completed hall roof lattice

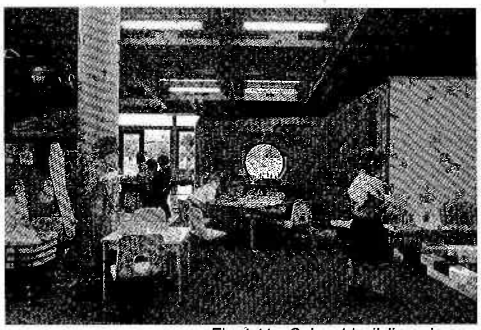


Fig 1.11 School buildings in use

After a roof section was assembled on the ground, it was lifted on top of the columns using a purpose-made lifting frame (Fig 1.9). A simple 25mm diameter stainless steel pin with split pins completed the connection. This procedure was repeated for each roof section in turn. When two adjacent sections were in place they could then be bolted together at the site splice (Fig 1.10).

The contractor followed this proposed method of construction with one adjustment. In order to avoid possible discontinuities at the site splices, it was agreed that all roof sections could be lifted in position and spliced before the roof was given its precamber. By jacking up the roof at a number of points and making the coach screw/sleeve connection in the lattices with the roof in this position, the required precamber could be achieved.

The completed Totton First School had its first intake of pupils in the autumn of 1990 (Fig 1.11). For the future we expect to explore further possibilities of glued laminated timber in new projects, particularly where prefabrication can be used to advantage.

Nils Den Hartog and Richard Harris

Project data

Client	Hampshire County Council
Architect	Hampshire County Council
Structural Engineers	Buro Happold
Services Engineers	Hampshire County Council
Quantity Surveyors	Hampshire County Council
Main Contractor	Louis Thompson (Southern) Ltd
Project Value	£1m
Completion Date	July 1989

Bishopstoke Infants School

Apart from the general plan, the depression of the building into the ground to reduce its overall height (Fig 1.12 a, b) is all that remains of the grass covered classrooms and central tepee originally conceived as the architectural solution by the late David White and his team at Hampshire County Architects.

The central hall of the school comprises a top lit drum 15m in diameter which generates the radial spiral grid of the nine classrooms and accommodates the administration areas (Fig 1.13 a, b). The classrooms look outwards beneath the perimeter eaves to the playground (Fig 1.14) which is protected from the external gaze of the surrounding housing estates on the south side by earth mounds. These are formed from the spoil excavated to found the building on a single raft directly onto clay-bearing strata. Light is introduced into curved internal corridors via a patent glazing strip between the load bearing block walls of the hall and the classrooms. This glazing is supported by laminated timber beams and minimal metalwork. The load bearing walls wrap around the north side of the hall to contain the community facilities overlooking the woods through smaller, north-facing windows.

The complex geometry and concomitant architectural requirements for this project led to the early decision that for costs to be kept at a reasonable level, the structure must be made simple and must utilise methods and materials which were malleable, allowing ready adjustment-to-fit during construction. For these reasons, in situ reinforced concrete floors were chosen, with load bearing blockwork, masonry walls and steel columns supporting a timber roof. Overall structural stability is ensured by utilising the layout of the walls and the plate action of the roof.

The sunken site

The site is underlain by a moderately firm clay into which conventional strip and pad footings would have to penetrate by up to 2m in order to achieve adequate capacity. However, the number and layout of such footings under each wall and column would have created a maze of trenches and used a considerable volume of concrete. It was therefore decided to found the main ground slab 1.5m directly into the clay, using it as a semi-raft to support all the walls and columns above. This helped the architect achieve the low level at the centre of the building that is essential to its section, and also provided the spoil from which the landscaping mounds are formed.

The ground slab has a downstand edge-beam around its perimeter to form a frost curtain, with a

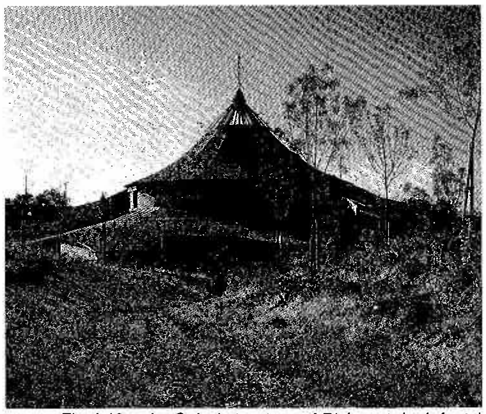
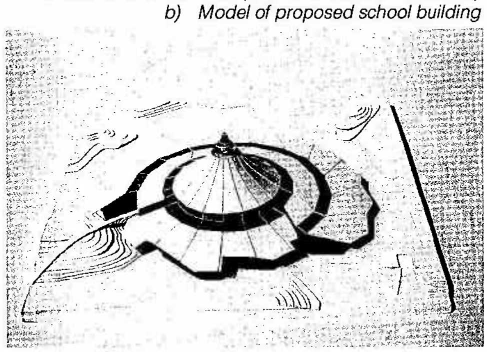


Fig 1.12 a) Spiral structure of Bishopstoke Infants' School in its sunken site (Credit: Jo Reid & John Peck)
b) Model of proposed school building



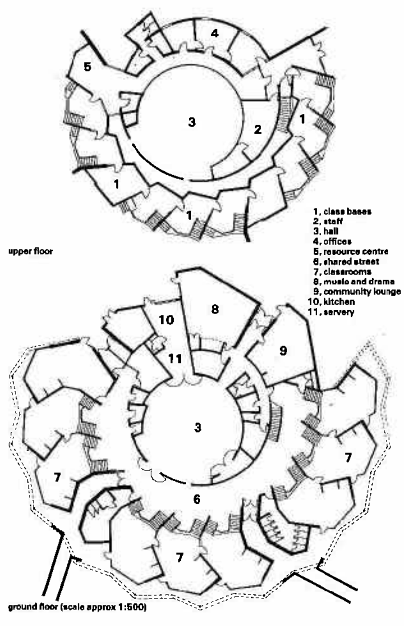


Fig 1.13 a) b) Ground and first floor plans showing classroom and administration areas

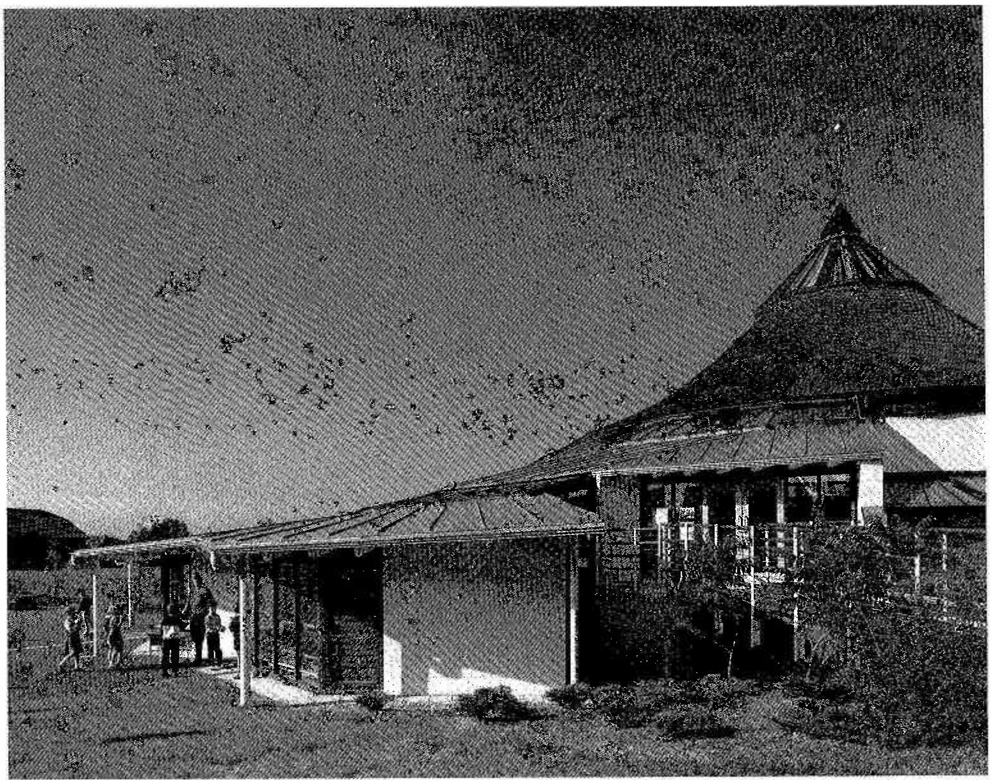


Fig 1.14 Classroom looking out beyond perimeter eaves onto playground (Credit: Jo Reid & John Peck)

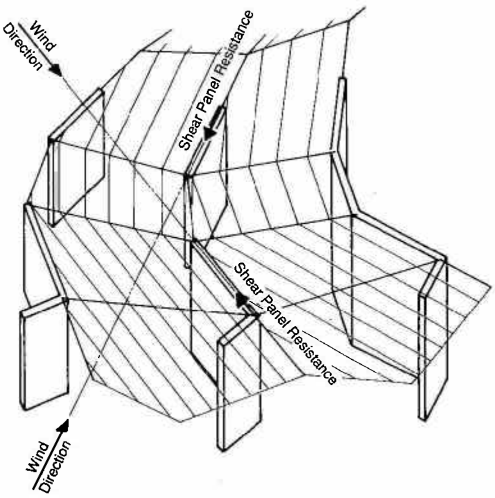


Fig 1.16 Plate action of roof

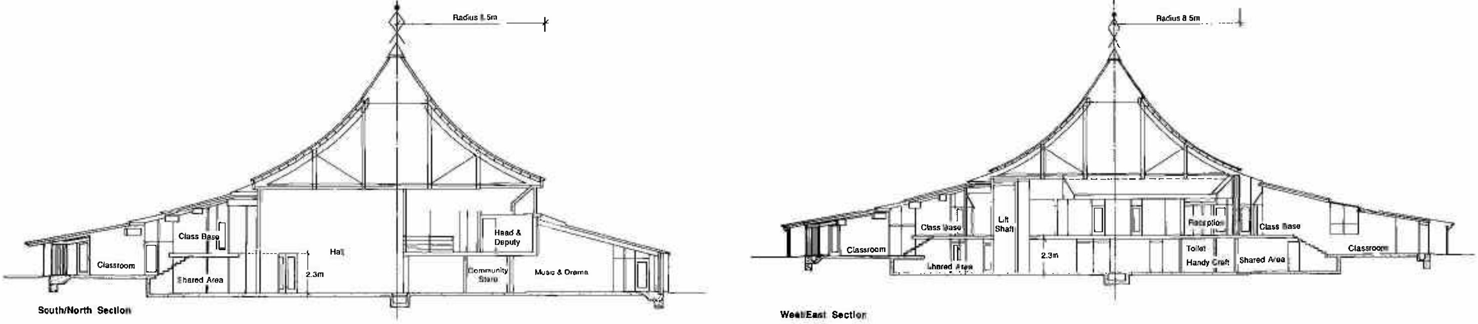


Fig 1.15 a) b) N/S and E/W sections through conical structure of school

150mm deep by 100mm wide duct let into the top of it to carry the heating and water pipes, thus avoiding the cost of a finishing screed (Fig 1.15 a, b). This, combined with the variety of different wall thicknesses, sill details and ground slab levels, led to over twenty different sections being required through the slab's perimeter.

A spiral structure

Almost all of the blockwork walls are load bearing, being of cavity construction where they have an external face, and single-leaf hollow-block elsewhere. Although the structure utilises only fairly simple house-building elements, examination of the plan for the blockwork walls shows these to have little continuity from which to derive overall lateral stability. To overcome this, reinforced concrete capping beams are incorporated into the tops of all the spiral walls, gaps in the spiral being bridged by steel box sections. Spirals are then linked together by the plate action of the whole roof. This action is generated by nailing a layer of plywood over all the rafters which are securely fixed to the rings of purlins. In this way, a lateral load from any direction and applied at any point can always be resisted by panel shear in at least one of the blockwork walls within the layout (Fig 1.16).

Steelwork in the project has been kept to a minimum so that the timber of the roof and screen partitions can dominate. However, the 200mm first floor slab for the classrooms is largely supported on

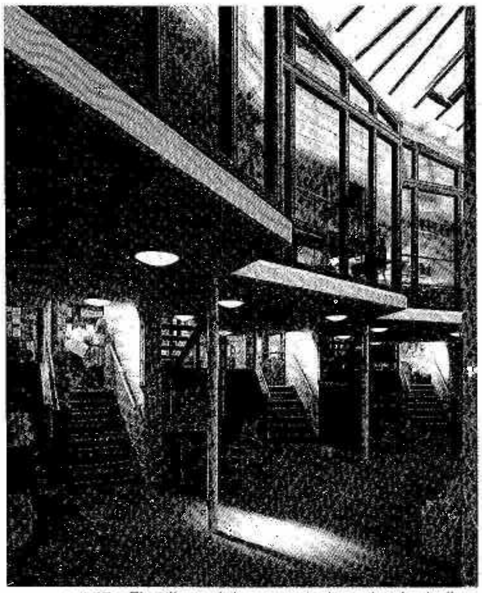


Fig 1.17 First floor slab supported on circular hollow sections (Credit: Jo Reid & John Peck)

circular hollow sections (Fig 1.17), these being not just attractive but, when protected with intumescent paint, offering the minimum size and shape to carry the required loads.

The long overhang of the roofs' eaves utilises two steel channels bolted back to back spanning between further circular stanchions. The depth of section was critical to the detail, for it supports the roofs' rafters from above and lies within the space taken up elsewhere by the insulation.

Roof design

The most striking structural feature of this school is its timber roof, an effect largely achieved by the building's geometry (Fig 1.18). From the glazed apex, the roof line curves down over the hall until it meets the spiral grids of the classrooms below. From there, the gradients become progressively shallower as the roof follows the spiral down to the widely-overhanging eaves. These lower roofs are supported on rafters at 600mm centres which run in sets parallel to each spiral grid line. These are in turn supported on regularly spaced concentric rings of glulam purlins which span between the blockwork walls or steel columns. The geometrical effect of combining these two grid systems is to produce some very complex shapes and planes, for although each ring of purlins is set to a single level, the length of each rafter between purlins progressively increases. Consequently the gradient of each rafter decreases, resulting in a gently warped surface that is in fact part of a helix (Fig 1.19 a, b).

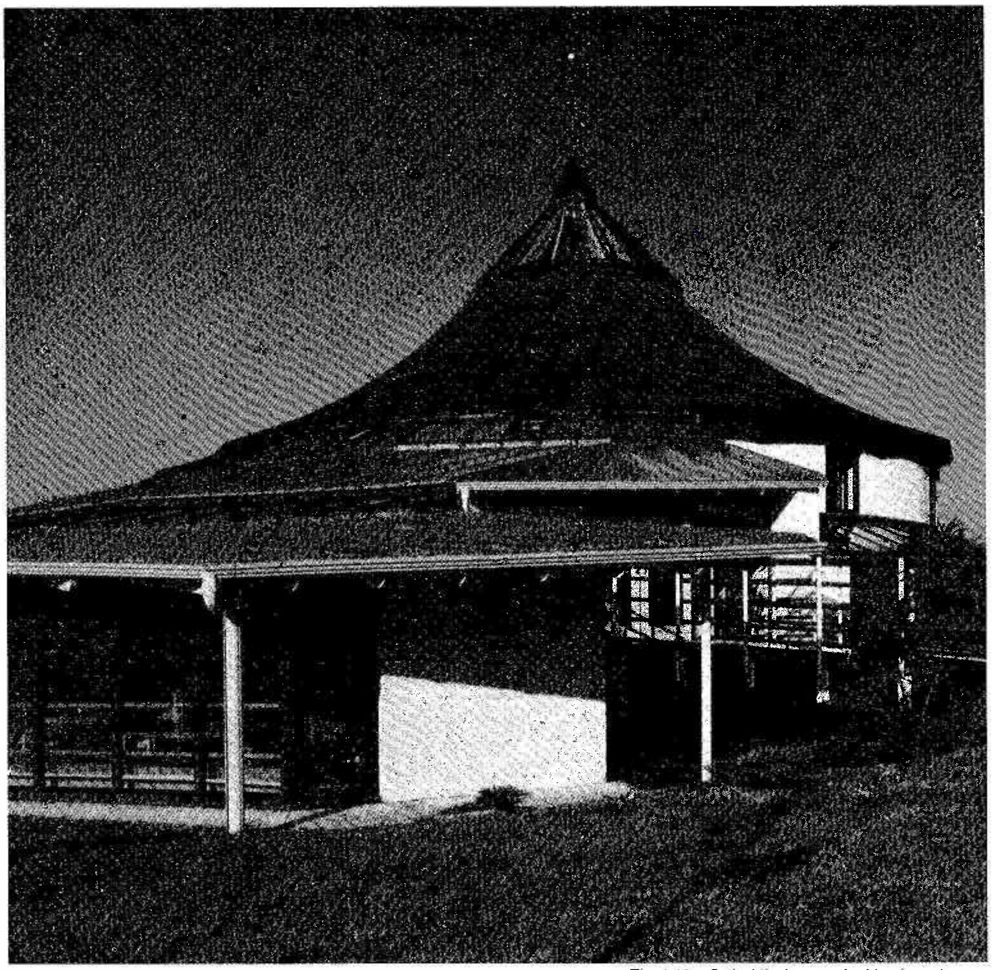


Fig 1.18 Spiral timber roof with glazed apex

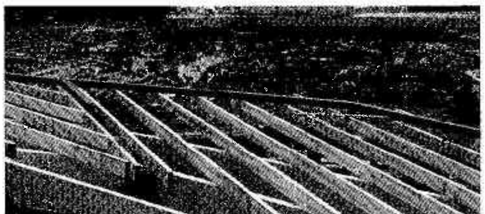
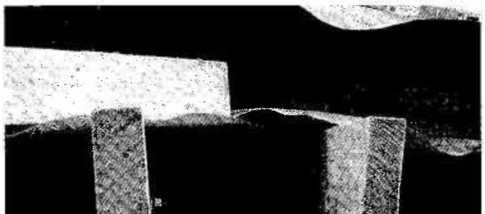


Fig 1.19 a) Rafters and purlins of timber roof



b) Detail of timber connections in roof

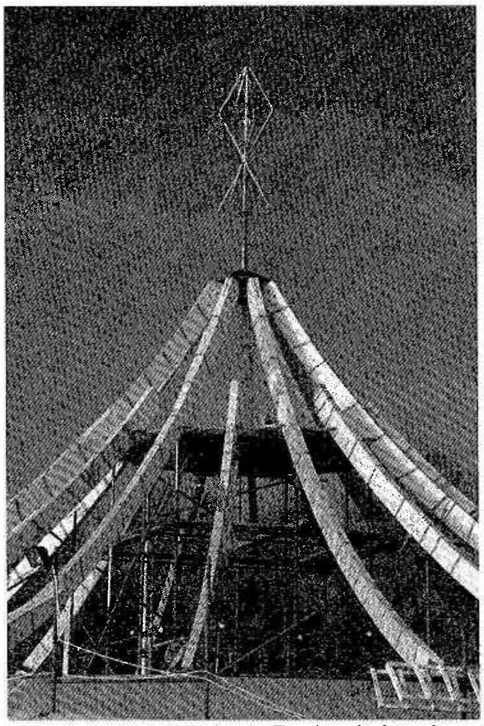


Fig 1.20 a) Erection of rafters of apex

b) Finished apex from within school (Credit: Jo Reid & John Peck)

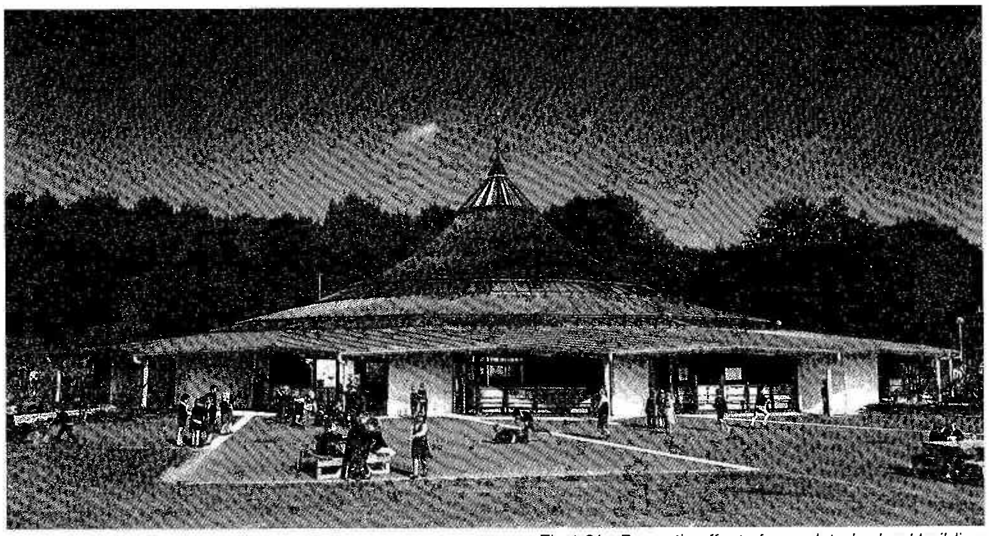


Fig 1.21 Dramatic effect of completed school building

The central roof over the hall is geometrically less complex but structurally more involved. It derives its shape from a set of eighteen curved glulam rafters, nine of which extend from the top of the drum wall to the apex and are strengthened by formation into a light truss of steel rods linked one to another. The intermediate non-trussed rafters also rest on the drum wall but only extend to two thirds of the height, their top ends being supported by a steel compression ring linking each to its neighbours (Fig 1.20 a, b). All rafters are joined at the base by a tension ring formed of a reinforced concrete capping beam tied down to the drum wall and hidden from view within trough-shaped lintel blocks.

The decision to extend the trussed rafters up to the apex was based on both aesthetic and practical considerations. Originally, a large air conditioning fan had been proposed in the apex, and all the glulam rafters had extended only to the compression ring at two thirds height. The fan however, had proved to be an unnecessary expense and steelwork used to define the apex was bulky and awkward. Consequently, it was suggested to the architect that the trussed rafters be extended, thus abolishing an unnatural break in line and a change of material, and also making the roof easier to erect as the principal load-bearing elements could then be locked into position at the apex. The original scheme had required the completion of the entire top compression ring before any of the glulams could become self-supporting and was therefore far less stable geometrically.

The rough timber rafters, exposed laminated timber purlins, radiators and fair-faced load bearing walls on a complex spiral grid offer a view of an infants school at Bishopstoke of low energy construction in its material content that is designed to be visually stimulating but complex to the user (Fig 1.21). However, such construction relies very significantly on the craft skills of the builder and the finished quality of this building is a tribute to its builders, Louis Thompson.

Ben Kaser and Mick Green

Project data

Client	Hampshire County Council
Architect	Michael Hopkins Architects
Structural Engineering	Buro Happold
Services Engineering	Buro Happold
Quantity Surveyor	Davis Belfield and Everest
Main Contractor	Wates Construction (Southern)
Contractor (Steelwork)	Tubeworkers
Contractor (Canopies)	Clyde Canvas Goods and Structures
Project Value	£630,000
Completion Date	December 1986

Fleet Velmead Infants School

Transferring from a stone and timber Victorian pile, cellular in plan, to an open plan metal and glass pavilion, could have been quite a shock for the headmistress and staff of Fleet Velmead Infants School, had there not already been a strong tradition of open plan schools in Hampshire. However, the even, low key but highly engineered appearance of the schools stems from the high quality of steel detailing and the integration of the well finished steel frame into the subsequent architectural elements.

Fleet Velmead Infants School sits on a fine, naturally wooded, sandy, flat site (Fig 1.22 a, b). Despite being constructed of unashamedly modern materials, the building and its playground tries to make the minimum impact on the heathland site which is itself regarded as an extension of the classroom, both in visual terms and as a laboratory (Fig 1.23). The outcome is the provision of full height single glazed panel windows and doors integrated into the steel frames of both north and south elevations. This visual connection to the outside has considerable benefits to the user but is accompanied by a slight loss of energy performance in the winter. However, the use of an elegant blue minimal fabric shading to the south elevation provides a counter to solar gain and glare in summer.

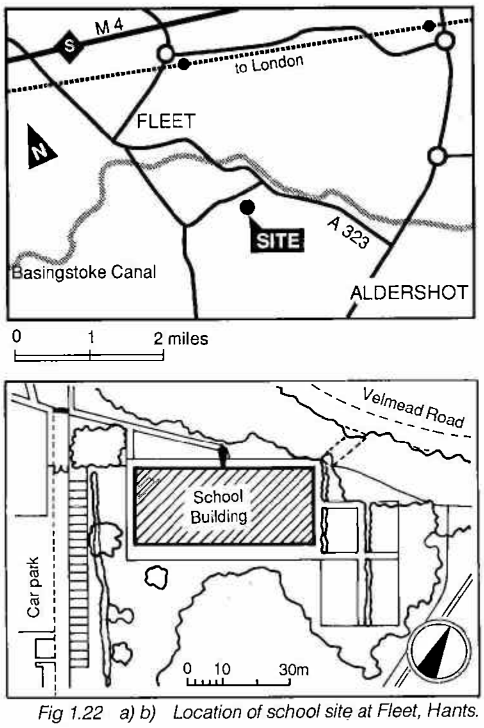


Fig 1.22 a) b) Location of school site at Fleet, Hants.

Fig 1.23 School building surrounded by woods and heath (Credit: Dave Bower)

Structural design

Indeed the first proposal for this school used a flowing membrane roof of PTFE glass fabric insulated where necessary, and organised within a 18m × 10m tubular steel grid (Fig 1.24 a, b). Such design would have been truer to the essence of the final 54m × 22m plan. In the event the structural solution was a large open barn section with top lit ridge, utilising a 3.2m wall height to the classrooms in order to accommodate the sports hall as well as all other facilities for teaching and administration under a common roof (Fig 1.25 a, b). With classrooms facing south, the music hall, teachers' rooms and hall are all housed beneath the sloping roof rafters to the north side with the dividing external glazed screens integrated into the structural steel grid.

In concept, this structure had its origins in those large French electricity pylons that support their high tension wires on outriggers. Here the tubular steel corridor frames of 138mm diameter/6.3mm tubes 2m wide support the roof rafters at 6m centres on 76mm diameter/5mm raking steel tubular struts, thus reducing the 10m span of the inclined roof rafters to a continuous two span beam of 2m and 8m. The rafter itself is a composite of 152mm × 229mmT welded to the top of a 76mm diameter tube so that the cold formed 170mm deep Z purlins can be hidden, sandwiched with the insulation between the grey colour coated profiled steel sheet roofing and the lower perforated profiled acoustic sheet sheeting (Fig 1.26). Only the 76mm diameter tube of the rafter is exposed below the ceiling line. At the join of the raking 76mm struts to the cranked composite roof, rafter stability is

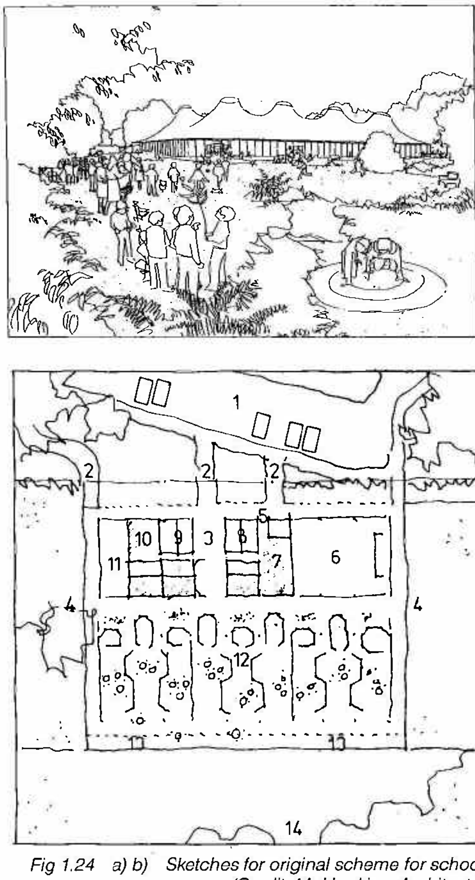


Fig 1.24 a) b) Sketches for original scheme for school (Credit: M. Hopkins, Architects)

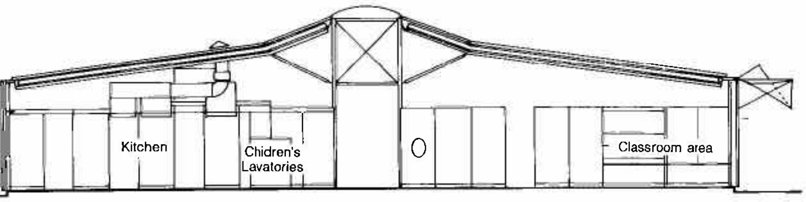
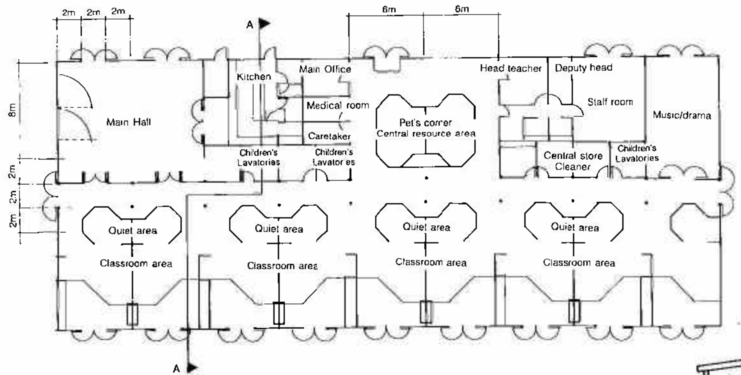
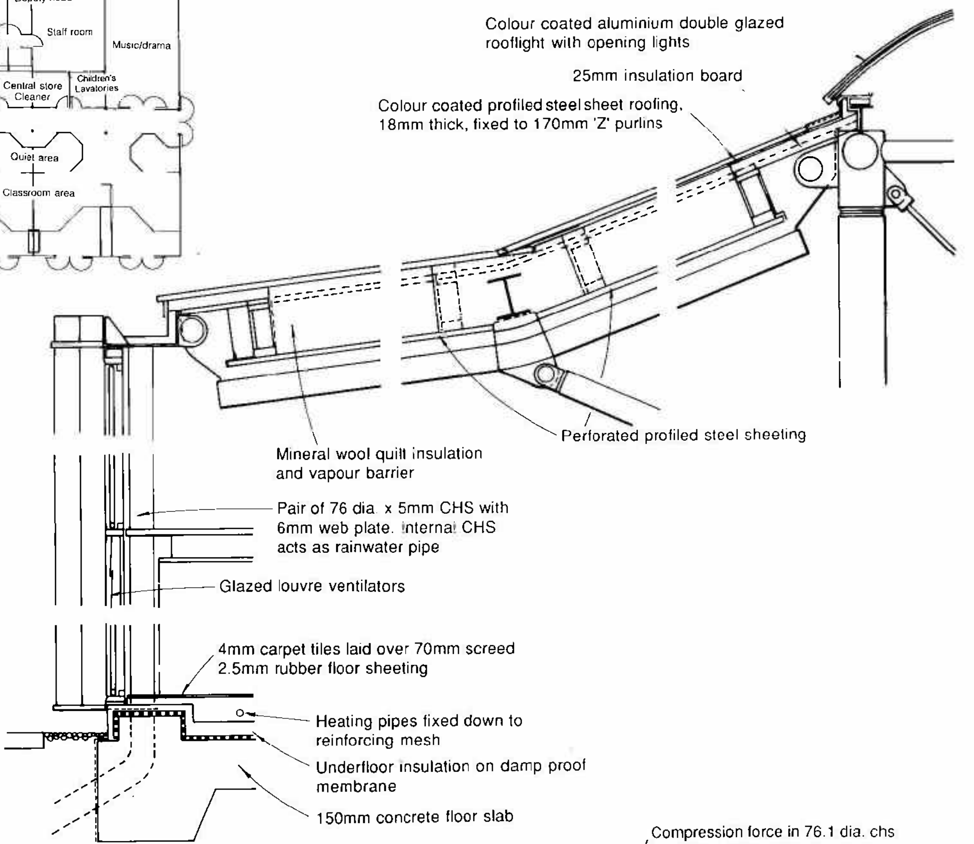


Fig 1.25 a) N/S section through school



b) Plan showing classroom and administration areas

Fig 1.26 Section through wall, eaves and ridge construction



ensured by replacing the Z purlin by a 127mm × 114mm × 26mm kg/m rolled steel joist to give the necessary twisting moment resistance.

At the external perimeter, the bottom 76mm diameter tube of the roof rafter stops short of the glazed elevation to allow the webb of the T section to be pin connected to the 89mm × 229mm structural channel. This forms the gutter and is supported by the external dumb bell columns formed by a pair 76mm diameter tubes separated by a 6mm webb into which the glazing frames could be fitted (Fig 1.27 a, b, c). Any effects of cold bridging are thus minimised. The form of dumb bell column enabled the rain water to be transmitted down through the inner tube also permitting wind and snow loads on the external shading fabric sails to be strutted off it.

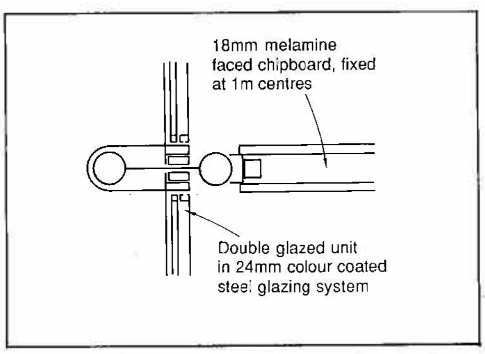
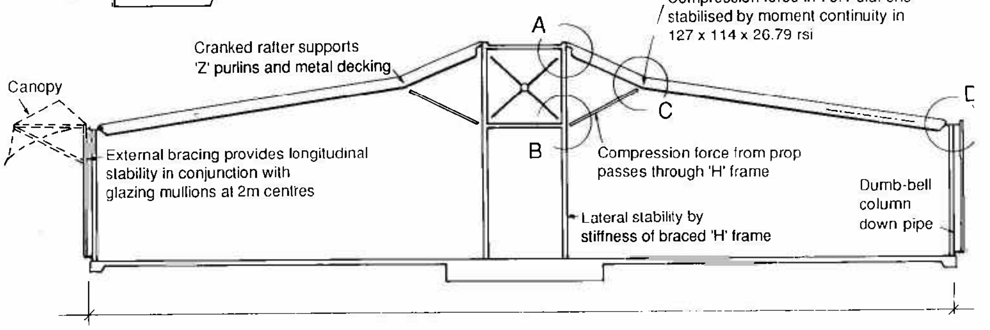
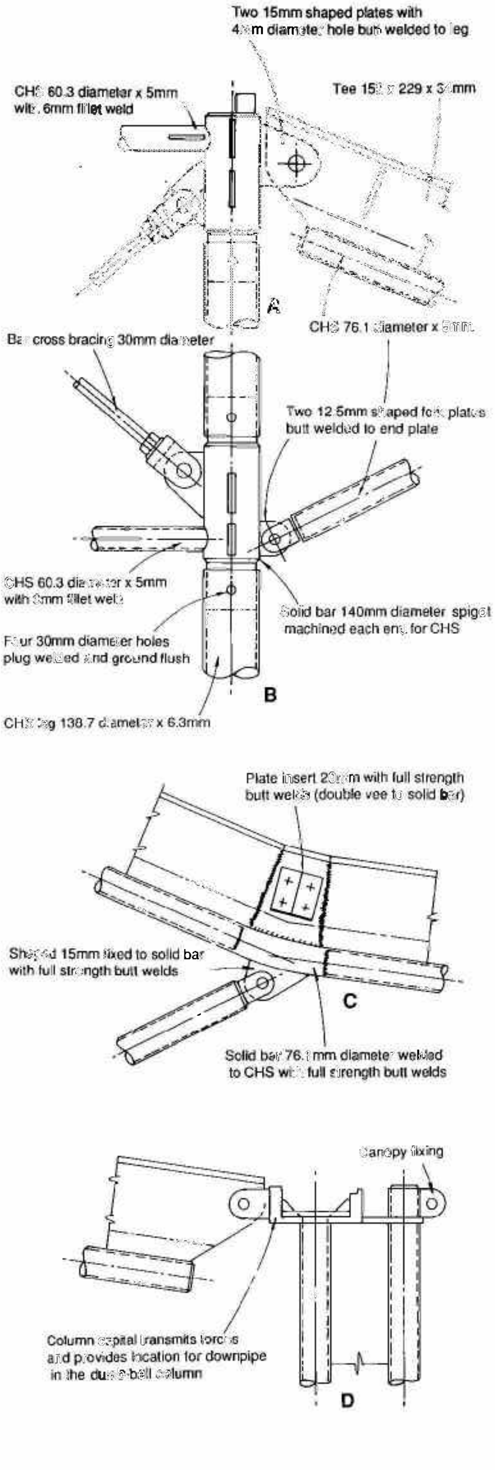


Fig 1.27 a) Dumbbell section through external column/internal partition junction



b) Structural section through buildin



c) Details of main structural junctions

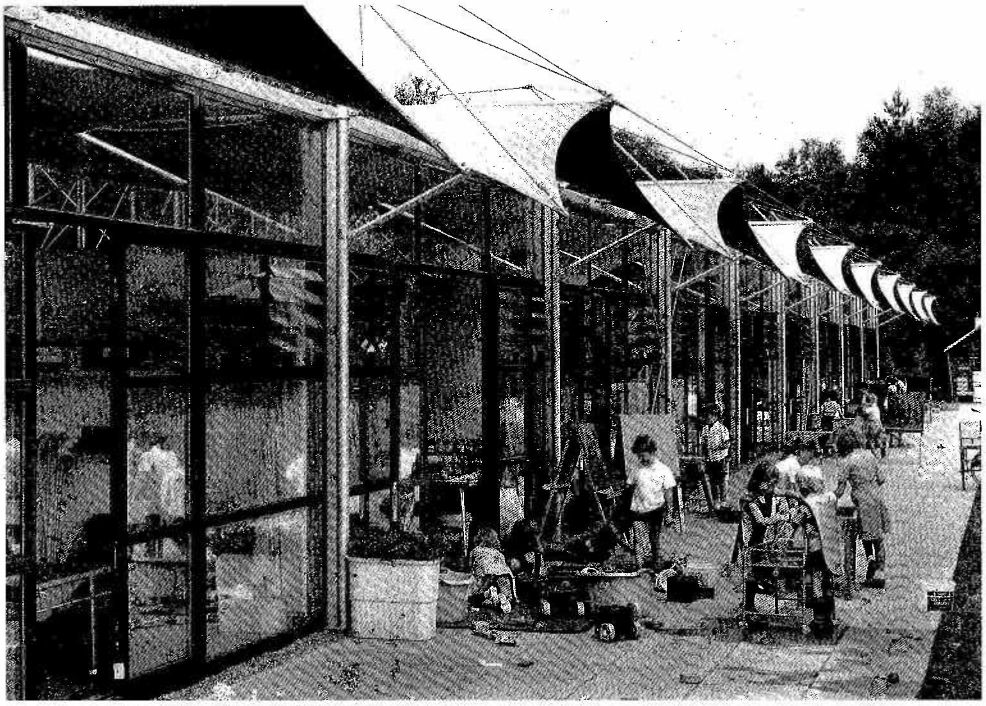


Fig 1.28 Stressed PVC sails forming solar shades over south-facing classroom perimeter (Credit: Dave Bower)

The 2m x 6m solar shades are formed from doubly curved sails of blue PVC-coated polyester fabric, stressed between brackets off the dumb bell frames by 6mm stainless steel rigging cables and a single central 40mm flying strut (Fig 1.28).

The lateral stability of the school is ensured by the stiffness of the braced H corridor frames in conjunction with their foundations (Fig 1.29). These are themselves stabilised against twist by the longitudinal high level tubular gate frames either end of the corridor. In the longitudinal direction, these gate frames join all 12 H frames, and together with the exposed cross bracing on the north and south elevations provide resistance to all longitudinal loads.

Detailing of the structural steelwork was considered important to fully articulate to the user the purpose of each structural part. The 30mm cross bracing to the H frames was connected by adjustable bolted fork end to single 15mm shaped plates on the 139mm diameter columns. The direct tension and compression forces from the composite action of the roof rafters and raking struts was transmitted through solid 140mm bar which was spigot machined at either end and fillet welded to the 139mm diameter tubular columns. The notch between the two was exposed in order to architecturally express their presence, and required the use of additional 30mm flush ground plug welding between the spigot and the main tube to ensure the full moment capacity of the H portal frame as the principal lateral support element (Fig 1.27).

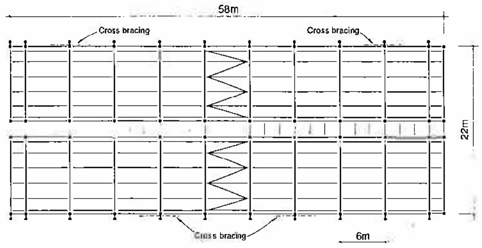


Fig 1.29 Steel framework plan showing braced H frames and cross bracing on N and S elevations

Foundations

The lightness of the building enabled all foundations to be mainly thickenings to the 150mm mesh reinforced slab even on this loose, sandy site. The ground slab has both a 1000g polythene membrane beneath it and a painted damp proof membrane on its top surface before receiving the insulation, heating coils, mesh and screed for the underfloor water heating system pipe coils to the standards required by Wirsbo's Agrément Certificate.

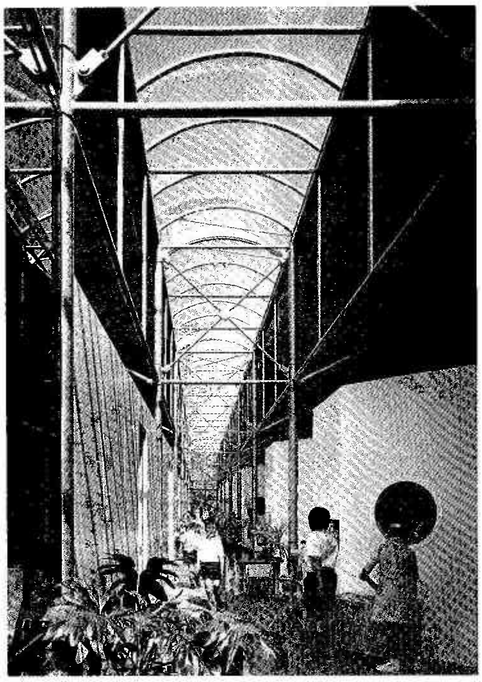


Fig 1.30 *Top glazed central corridor allowing light deep into school (Credit: Dave Bower)*

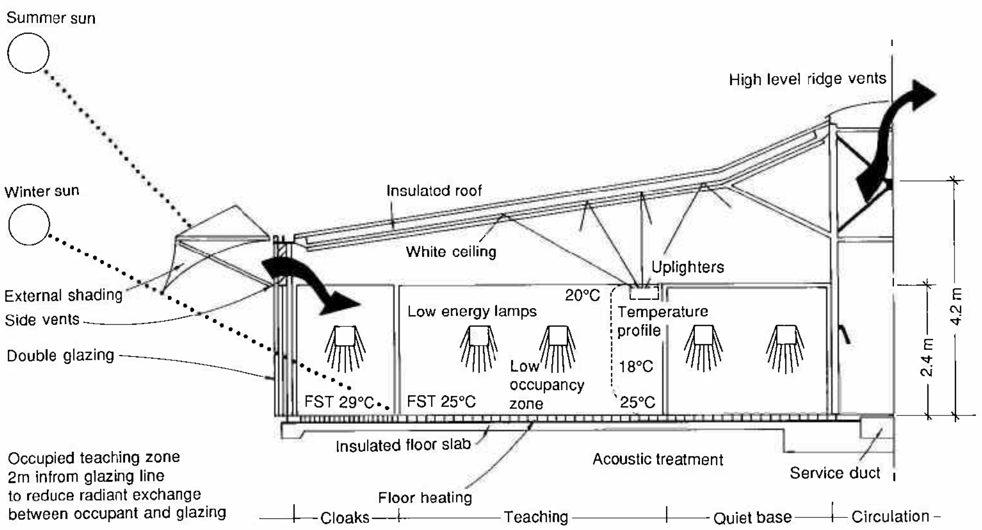


Fig 1.31 *Environmental section showing solar shading and ventilation patterns*

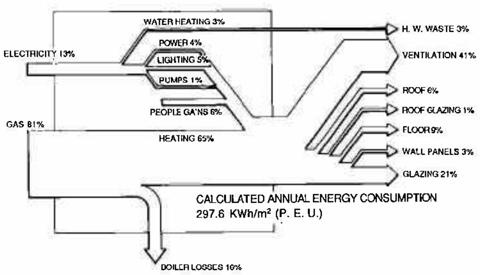


Fig 1.32 *Sankey diagram detailing energy consumption of school*

Building services design

In contrast to the high expressional form of the structure, the services and energy systems were designed to make the minimum visual impact possible on the internal working spaces of the school. A major aim of the design team was to optimise the total environmental quality of the working space – light, shade, heat, fresh air, views and spaces which stimulate and raise the awareness of the children to their surroundings.

The school was fortunate in being provided with an excellent wooded site which is bounded on the north side by tall evergreens sheltering the school from northerly winds and Velmead Road. To the south and east is natural heath land with tall deciduous trees beyond (Fig 1.23). It therefore seemed a natural decision to locate the teaching spaces to the south, with unrestricted views of heathland from the classrooms. Administration offices and the main halls are located on the north. A central corridor orientated east/west is top glazed, allowing light to penetrate deep into the heart of the school (Fig 1.30).

In placing the classrooms on the south-facing side and using a high percentage of glazing to maintain the views, care had to be taken to prevent overheating in the summer and cold radiation in the winter. To overcome these potential environmental problems, external shading devices were used to block out the high angle summer sun, yet allow the lower angle winter sun to penetrate into the classrooms (Fig 1.31). In order to prevent cold radiation between the glass and the children in winter, the teaching spaces are pulled back 2m from the glazing line, the 2m band accommodating wet activities and cloak areas.

The resulting glazed elevations and the central spine glazing provide excellent daylight quality within the school, artificial lighting not normally being required during the school days.

Energy efficiency

Optimum energy efficiency had to be sacrified to the total internal quality. It was still accepted however, that all necessary measures should be taken to achieve what could be regarded as a reasonably energy efficient building – all built within the construction budget and conforming to, or improving upon, the DES energy guidelines set out in DES Design Note 17 (Ref 1.1). The Sankey Diagram (Fig 1.32) illustrates the school as having a calculated annual energy consumption of 297.6 kWh/m² (PEU) – DES Guide Note 17 sets a target of 304 kWH/M².

Selection of design critiera involved striving to achieve a balance on the total energy systems, so compensating for the lack of performance in any one detail. All external opaque areas and floor slab were therefore insulated to a high standard (Table 1.1), with the aim of compensating for the extensive glazed elevations.

Table 1.1 Thermal properties of materials

Item	'U' Value W/m²C	Admittance W/m²C	Time Lag (hours)
Roof/wall panels	0.3	0.44	1.2
Glazing	3.6	3.6	–
Floor	0.27	3.0	4.0
Internal Partitions	1.17	1.65	1.6

Thermal classification of building: lightweight with a response factor of 1.6.

A considerable number of analytical checks were carried out to determine the environmental performance of the school in the summer, as the high percentage of glazing and the lightweight response factor of the school had drawn attention to some potential environmental problems. Calculations using the CIBSE Admittance method produced some worrying results. However, it was felt that the external shading, volume and height of the internal spaces, effective ventilation of the cross section and the non energy storing lightweight roof and walls would create an environment capable of controlling potential summer problems. Reasoning has been validated as the school has been able to monitor perfectly acceptable temperatures during the warm summer months.

Heating, ventilation and electrical provision

The school is entirely heated by an underfloor system of water coils giving an even distribution of heat throughout the occupied spaces. Heat is low grade and radiant, such a system being ideal for small active children, who spend a majority of their school day sitting on or being near to the floor. Radiators were discounted because of the possible poor distribution of heat through the open plan of the school. A forced air system was also considered, but also rejected because of potential fan noise and the risk of hot air rising to roof level. Both systems could have resulted in a greater calculated heat loss and annual energy consumption than the underfloor system selected.

Two natural draught gas fired boilers generate heating water at 80°C, which is then reduced to a flow temperature of 45°C for the underfloor heating system. Temperature drop on the primary system is kept at 10°C, maintaining the return temperature to 65°C at maximum load to prevent back end corrosion on boilers. Temperature drop on the secondary underfloor heating system is 5°C at maximum load conditions. System controls on the central plant include optimum start and weather compensation.

The underfloor heating is divided into seven individually controlled zones, each designed to produce a resultant temperature of 19°C at −3°C outside. The underfloor circuits are constant temperature (depending on the degree of external compensation) with variable water volume. Within the main body of the building the design floor temperature is 26°C whilst in a 2m band around the perimeter of the school the design floor temperature is 29°C. This increase in floor temperature is achieved by reducing the spacing between water coils.

The school is naturally ventilated, with the exception of the internal toilets and kitchen area. The cross section of the building is ideally suitable for cross ventilation, with air inlet via glazed louvres at the edge and exhaust via the high level ridge vents (Fig 1.31).

Electrical services to the school are standard, distribution being broken up into three main west, east and kitchen zones. The artificial lighting is discreet and uses low energy lamp sources. Only in the main reception/resource area are tungsten lamps added to give sparkle to the space.

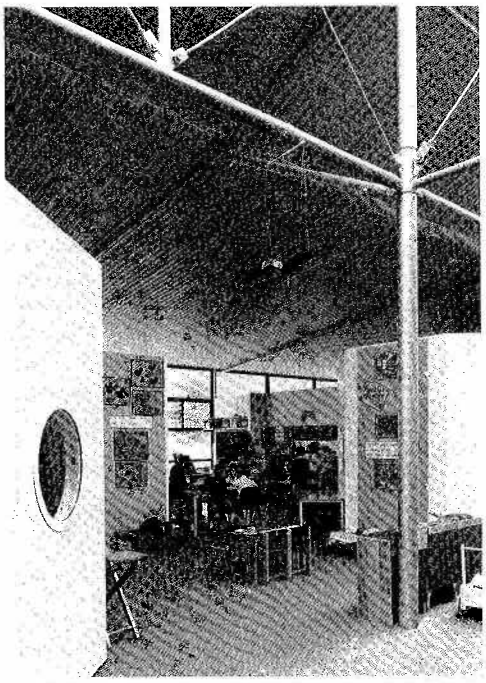


Fig 1.33 Open plan layout of classroom interior (Credit: Dave Bower)

Classrooms are lit by metal halide uplighters which are reflected off the white sloping ceiling (Fig 1.33). However, in practice it has been found that reflection from the ceiling does not match design figures – a consequence of the acoustic material, perforated for sound absorption. It is however, a successful lighting scheme which supplements the visual environment of the spaces, highlighting the structure and creating subtle areas of contact on the ceiling slopes (Fig 1.34).

The desire to maintain uninterrupted clear spaces and clean lines required a substantial effort in the planning and detailing of services distribution. Thirty two services drawings were produced, well above average for such a building.

Users' reaction has been favourable – the teachers and children enjoy the high level of natural light in the school, the visual contact with the outside and the thermal comfort of the underfloor heating system. Since completion the school has attracted much critical acclaim, receiving the 1987 Steel Construction Award for single storey buildings. Indeed, it appears as an example of steel construction on Professor Lobster's (Ken Martin of the University of Liverpool) schools programme on 'Materials for Construction' broadcast by ITV.

Michael Dickson and Tony McLaughlin

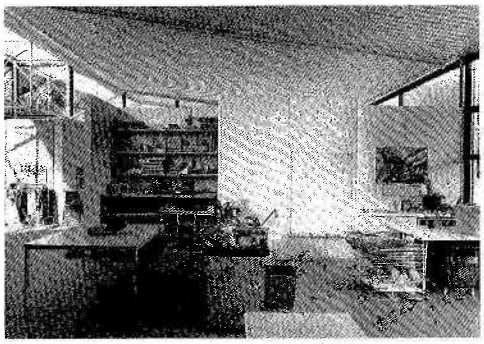


Fig 1.34 Typical class area with uplighting reflecting off ceiling slopes (Credit: Dave Bower)

References

1.1 Department of Education and Science. Architects and Building Branch: 'Guidelines for environmental design and fuel conservation in educational buildings' Design Note 17, 1981.

The State Mosque of Sarawak at Kuching, Borneo

Project data

Client	Pengarah Kerja Raya
Architect	Sami Mousawi International/Perunding Utama
Structural Engineers	Buro Happold
Services Engineers	DSSR
Quantity Surveyors	Widnells Trollop/KUBS
Main Contractor	Ssang Yong Worldwide Construction
Subcontractors	
(Precast Concrete)	Structural Concrete Shd Bhd
(Structural Timberwork)	Yu Sung
Project Value	£8m
Completion Date	October 1989

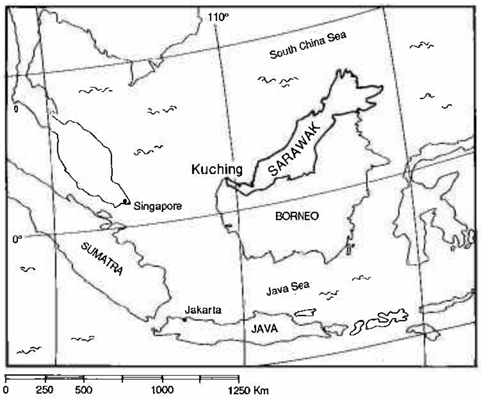


Fig 2.1 Location of Kuching in Sarawak, Borneo

Sarawak is a state of modern Malaysia situated on the northern side of the island of Borneo (Fig 2.1). For 100 years from 1840 to 1940 the country was governed by three generations of Brookes – the 'White Rajahs' who in an idiosyncratic, liberal style attempted to control and organise the various indigenous tribes, both dissuading the Dayaks from head hunting and controlling the Chinese traders.

After the Japanese were removed in 1945, Sarawak was a British Colony for about ten years before becoming part of the new federation of Malaysia. The tribes had adopted a range of Christian sects, largely owing to the Brookes' policy of evenhandedly dividing the country territorially amongst the missionary groups. The Malays however, were Muslims and by virtue of their Bumiputra policies have attained political ascendancy. Although the country remains officially mixed in race and religion, Islam is increasing and State mosques have been built in other states. The State Mosque of Sarawak was the dream of the Tun, Abdul Rahman.

After some consideration of designs by local architects, Sami Mousawi, an Arab by birth, who was trained in Germany and has subsequently lived in Manchester for 20 years, was commissioned to prepare a design for a mosque and cultural centre. He has executed a number of notable buildings, mostly in Islamic style, the most prestigious being the Mosque of Rome. Buro Happold was appointed as structural engineers for the Sarawak scheme and a further association with local architect/engineering firm, Perunding Utama, was established.

The concept for the whole complex was developed to scheme design stage, where the 90m × 90m mosque was the central building of a 500m × 500m cultural centre (Fig 2.2). This being a political building, the design brief changed with political fluctuations. In the event, only the mosque, riwag and minaret went forward to detail design and only

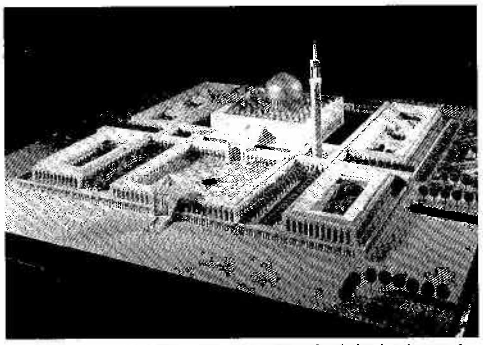


Fig 2.2 Model of mosque at centre of original scheme for cultural centre

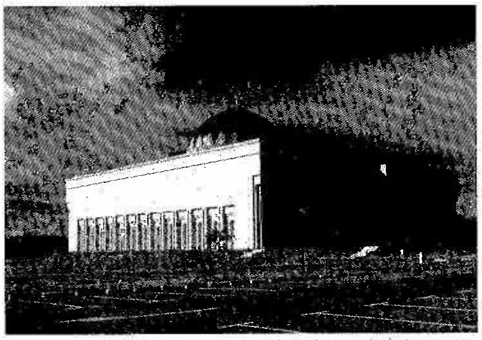


Fig 2.3 Exterior of completed mosque

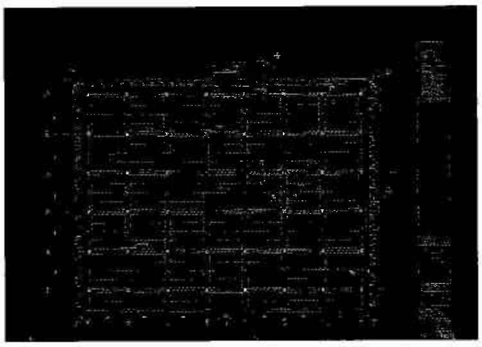


Fig 2.4 Floor plan of mosque

the mosque itself was finally constructed with a temporary arrival and ablutions area (Fig 2.3).

Structural design

The mosque is essentially a square box 90m × 90m on plan and 10.8m to the general ceiling level (Fig 2.4). The walls have a total thickness of 1.8m at ground level, but are heavily modelled with both the external and internal surfaces stepping in and out to form deeply recessed window openings. Structurally, the walls are in reinforced concrete, clad externally with reconstituted stone made from

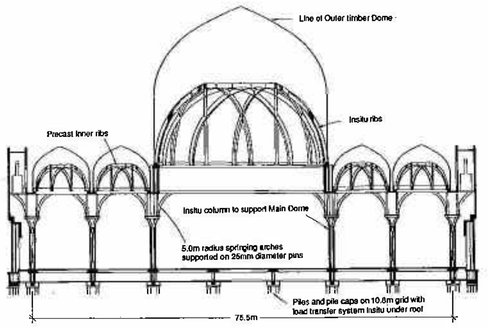


Fig 2.5 Section through mosque dome, walls, columns and floor slab

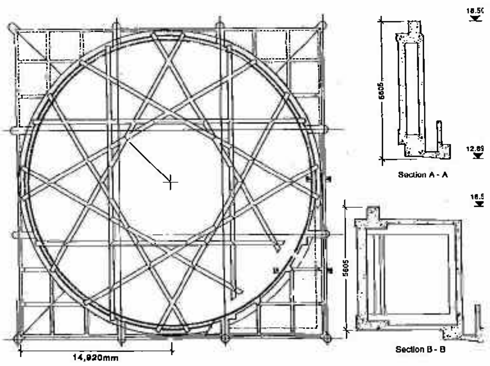


Fig 2.6 Plan and sections through ring beam slab

the local white limestone aggregate and white cement.

The roof is supported on complex column clusters on a 10.8m grid, there being eight rows of columns in each direction. Each column cluster consists of four columns rising from ground floor level and curving outwards to a collar 9m above (Fig 2.5). Standing on the collar are eight flying half-arches which support the roof slab. The air distribution was to be via a plenum space in the undercroft, entering the hall through the space between the four parts of the column clusters. Air conditioning was abandoned as part of a cost saving exercise but the plenum and plant room spaces remain.

The roof slab is in reality a series of interconnected ring beams, since each 10.8m grid square accommodates a 10m diameter opening for a dome, and each column cluster supports a square pyramidal rooflight. The soffit of the slab is also modelled with circular patterns of steps. The general thickness of the ring beam slab is 500mm (Fig 2.6).

In the centre of the hall but displaced one grid to the west, in the direction of Mecca, four column clusters are omitted and a 30m diameter opening is left in the roof slab for the main dome. The 12

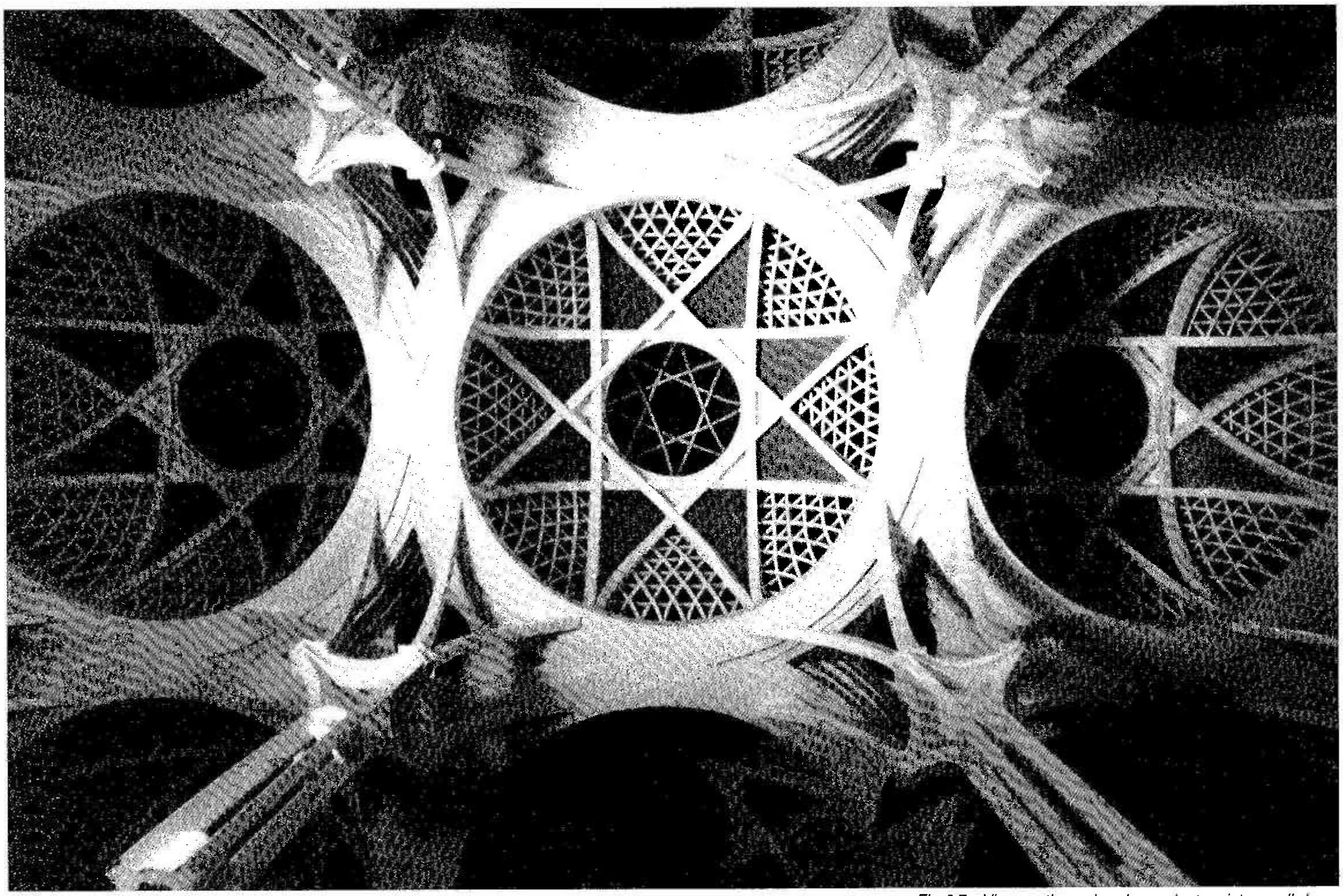


Fig 2.7 View up through column clusters into small dome

column clusters around this opening are each strengthened by a central 600mm diameter concrete shaft to support the weight of the main dome.

Lateral stability of the 90m × 90m roof slab is provided by the external walls acting as shear planes. The roof slab itself acts as a plate, transferring horizontal forces to the side walls. However, as both roof slab and walls have many holes and windows, it was difficult to ensure that these load paths were adequately maintained.

Each of the 40 small domes consists of both outer and inner dome areas. Each inner dome, made up of eight intersecting concrete ribs with plaster infill panels, has at its centre a 3.5m upper inner dome of GRP with a decorated soffit (Fig 2.7). The outer domes are constructed in timber, each having 32 radial ribs laminated to the profile of the dome. Four rings of horizontal beams are overlain by a system of diagonal lattice bracing members, and the whole structure is covered with boarding and clad with copper. The larger main dome is similar in structure but has an inner dome with 12 intersecting ribs springing from 24 points. The concrete ribs are similarly infilled with plaster and tracery. The 10m diameter upper inner dome is constructed in timber with a decorated plaster interior shell. The external dome, 30m diameter and 25m high, is constructed with 48 ribs, five rings of beams and lattice bracing. Again, the outside is boarded and clad with enamelled copper plates.

The weight of the main dome is transferred to the 12 surrounding columns by a 5m deep concrete torsion box which has internal shear walls onto the surrounding columns.

Foundation difficulties

At the beginning of the project a preliminary site investigation, requiring 49 boreholes on a 60m grid, was undertaken over the whole 25 hectare site of the cultural centre. Layers of peat and silt were found to overlie Bau limestone, which varied in depth from 5 to 30m. In one borehole a curious weathered limestone was encountered, for which no local explanation could be found. After some consideration large diameter bored piles socketted into the limestone were chosen for the foundations.

The Bau limestone is named after the nearby town of Bau, where it outcrops revealing cliffs, caves and large fissures. Concern grew that such features could occur below the site and it was decided that all main internal pile groups should be probed with a rotary core sampling drill, 5m into the limestone to

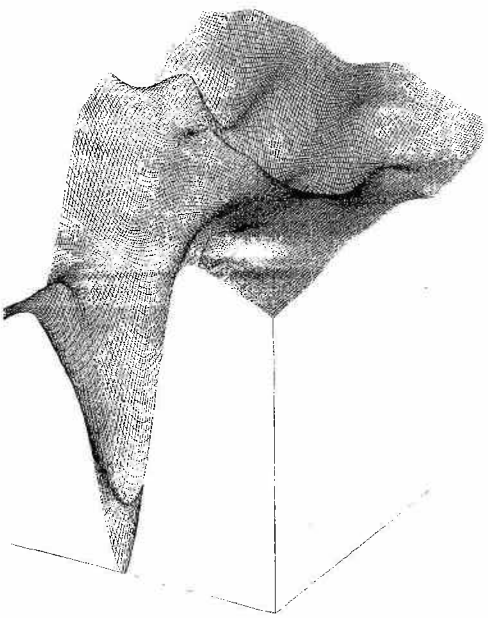
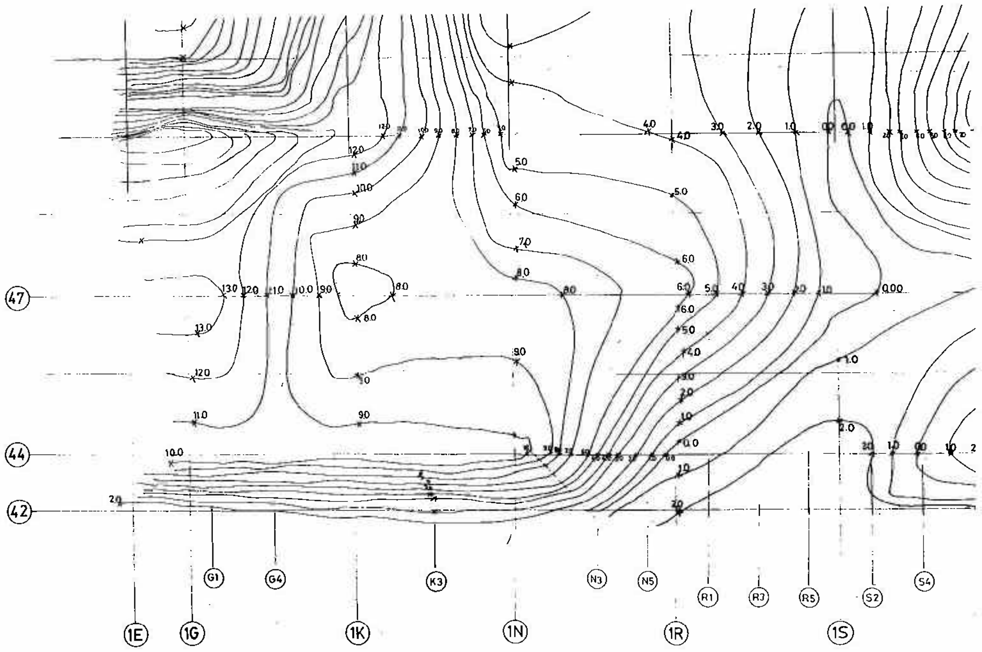


Fig 2.8 a) Computer generated image of rock surface topography



b) Interpolated rock contour ma

check for cavities. In carrying out this exercise considerable fluctuations were observed in the rock head level, cavitations up to 3m into the rock strata were found and an area was located where dykes of an igneous intrusion had affected the local rock composition. This intrusion had altered the rock strata over a quarter of the plan building area producing marbles, limestones and igneous material weathered to varying extents to great depths.

These sudden undulating variances in both founding formations levels and bearing strata resulted in the need for additional works. A rotary percussive survey was carried out at all pile locations and at 1m offsets to map the rock formation against adjoining levels. In certain locations this resulted in the use of extended rock sockets. In areas of highly weathered strata 5m rock sockets were adopted and tested so the pile performed in end bearing and skin friction load transfer. Where cavitations existed the pile formation level was extended below these and infilled with concrete until a standing head of concrete existed in the temporary lining casing. Finally at five pile group locations rock level differences of up to 12m occurred caused apparently by pipes and cliffs. In these cases, after thorough level probing established rock contours the piles were located, pile caps redesigned and gullies were infilled with mass concrete to stabilise the rock mass.

To guard against possible pile failure caused by cliffs, fissures, cavitation and the geological variations in rock type, design of the undercroft structure was revised from the original tie beams between pile caps to a system of linear reinforced concrete cross walls for the full depth of the undercroft (Fig 2.8a,b). These could span between pile groups and redistribute loads caused by a failed pile.

Piles were placed by driving temporary casings, boring to remove peat layers, followed by socketing into the rock by chiselling and coring.

Concrete frame construction

Following the completion of pile placing, work then proceeded on the frame requiring both in-situ and precast concrete construction.

In-situ work

A concrete batching plant was set up on site, the mix constituents being highly fragmented local crushed-limestone, washed river sand and cement imported from Western Malaysia. The concrete was dry batched into wagons, water added from the plant and the constituents mixed in the drum of the wagon, then placed by skips off the three site tower cranes. An on-site concrete testing laboratory was set up to ensure concrete quality, and cube strengths in excess of Grade 30 were achieved with the fairly high-slump plasticised mix.

The ribbed ground floor and external wall columns were constructed with steel or timber formwork up to ring beam level at +9m. Reinforced concrete walls above the ring beam were constructed in two 3m high lifts, while the roof slab was constructed on a massive temporary works erected off the ground floor to +15m level. The 40 small dome rings and the main dome 5m deep torsion box were cast off this, the main dome inner ribs being cast in sections on extensive scaffolding erected to 35m height from the ground slab.

Precast work

All internal columns, arches, infill ribs to small domes and stepped roof lights were of precast concrete. A precasting bed was set up on site and columns and arches cast in batches of eight units per day, while the legs and central hexagon of the domes were formed individually. The cruciform 'hockey stick' columns were cast as four units with insitu stitches at ground, mid-height and capital level formed by steel shutters clamped in position once the column elements had been erected and plumbed into position. The anchorage requirement of the reinforcement at the capital with inserted pin supports necessitated a very complex arrangement of bars.

Once the column stitches to capital level were complete, the eight radius arches were lifted and held in position by a steel temporary works frame which allowed adjustment in arch angle. The formwork for the roof deck was then erected and

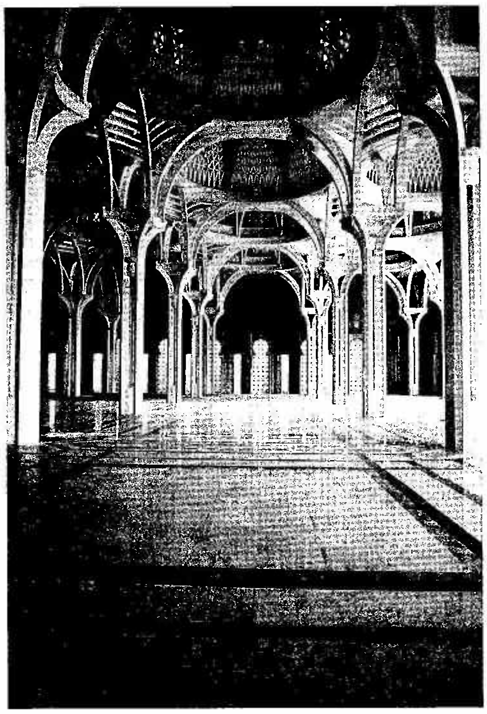


Fig 2.9 a) b) c) Erection sequence of columns, arches and ring beams of concrete frame

the ring moment steel installed (Fig 2.9 a, b, c). The arches sit on a 25mm diameter levelling pin at the capital level, which was subsequently grouted in once the roof slab had been poured. The moment carrying reinforcement projecting from the arch was bent from a curve in section to horizontal, and from straight in plan to radial, to suit the ring beam geometry.

The precast ribs to the small domes were formed as six double sets of legs with a single hexagonal central ring, which was supported on a frame suspended on rails spanning the 10m hole. The legs were then positioned and clamped to the central section. The bottoms of the legs were cast into insitu stitches at the ring beam while the top was connected by welding loose plates across the joint, and finished with epoxy mortar.

Timber construction

Sarawak is unfortunately one of those countries where timber is felled from the tropical rain forests using heavy machinery, which so damages the ground cover that heavy rain washes away the soil, making it difficult for plants to become re-established. The Bumipatras sell the timber rights to the Chinese, who cut it down and sell it on to the western economies. Little of it is processed in the country. In an idealistic way, it was felt that the roof structure of this nationally important building should be fabricated from the local timber, rather than of imported steel. It was thus hoped that local industry might be stimulated to set up factories to produce structural timber buildings.

In the event, the Korean contractors had a lot of difficulty obtaining suitable timber, and mixed meranti had to be substituted for the dark red meranti originally specified. As there were no processing facilities in Sarawak, the timber was sent to Seoul to a factory which specialised in making high quality veneered doors. Although the Koreans had never attempted laminated or any other form of structural timber before, they were intrigued by its possibilities. A crude but effective hydraulic laminating press was set up and the locally available resourcing of resins was investigated. A finger jointing machine was also specially developed.

Work on the ribs for the small domes was undertaken first. With a 120mm × 60mm finished dimension, these were laminated from boards 25 × 65mm. The 25mm dimension was really too large for the bending radius of the ribs (4m) but surprisingly, very few breakages occurred. If such a breakage did occur during lamination the ribs were simply released from the bending press, the damaged piece replaced and the rib completed. One problem caused by the use of thick layers was that of uncurling of the finished laminate. It proved quite tricky to develop a satisfactory compensation shape, and because of the variance in curvature of ribs delivered to site from Korea a check bed was formed on site to assess shape and provide accurate locations for preboring connections. A certain deviation from the required shape was allowed, this being compensated by forcing and clamping the arch to its final shape on the site check bed.

Termites, both ground and flying varieties, are highly active in Sarawak, and like all wood-eating insects they consume sap wood with a high moisture content. Consequently all timber had been specified for vacuum treatment with copper chrome arsenate (CCA) salts. This could be carried out at a plant in Singapore but being a water-based process, time was required for the timber to dry out before laminating, proving to be too lengthy for the mosque programme. Solvent treatment could not be carried out before laminating for fear of affecting the gluing process. As a solution, the laminated timber was solvent treated by dipping after laminating, while the diagonal bracing and external boarding were CCA treated and air dried on site.

Steel connection plates were made up in small workshops in Singapore. One of these also stamped out the 'Bulldog' toothed plate connectors to British Standard patterns. Extensive testing of joints was then carried out at a local laboratory to

Fig 2.10 Varying stages in construction of timber framed main and small dome

assess performance.

The small domes were assembled by jig on site at ground level. The apex joint was strengthened so that the dome framework could be lifted whole and placed on the roof. Boarding and waterproofing were then completed in-situ (Fig 2.10).

The large dome was built in place using much scaffolding. Wetting and drying of the timber in the tropical rainstorms proved troublesome. When the timber dried it shrank, causing the bolts to loosen. These were then tightened up but the wetting of the timber caused the wood to expand and yield the bolts, which again became loose after further drying and shrinkage. After a few bolt breakages,

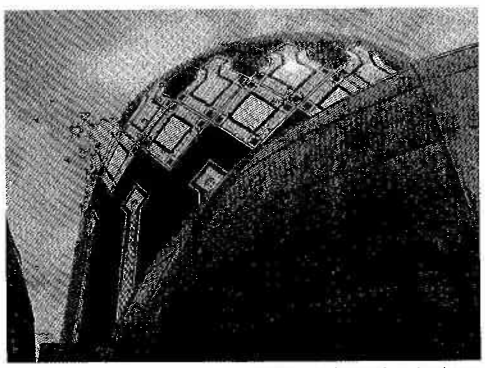


Fig 2.11 Finished outer surface of main dome

retightening was restricted to one operation before the dome was boarded. It was then immediately waterproofed (Fig 2.11).

In presenting this design and construction it is not only relevant to review the ingenuity and innovative nature of the structure developed by the consultant team, but also to congratulate the contractor and his subcontractor for their resourcefulness and practical determination to progess these highly demanding and complex construction works, with a predominantly local workforce, at this fairly isolated location.

Ian Liddell and Mike Shaw

Pinned or rigid frame design?

Project Data	Buckingham Hotel
Client	Naaz Investments
Architect	Iain Pattie Associates
Structural Engineers	Buro Happold
Services Engineers	Buro Happold
Quantity Surveyors	Buro Happold
Main Contractor	Try Construction Ltd
Project Cost	£11.7m
Completion Date	1991

Fig 3.1 a) Artist's impression of refurbished Buckingham Hotel facade (Credit: Iain Pattie Assoc. Architects)

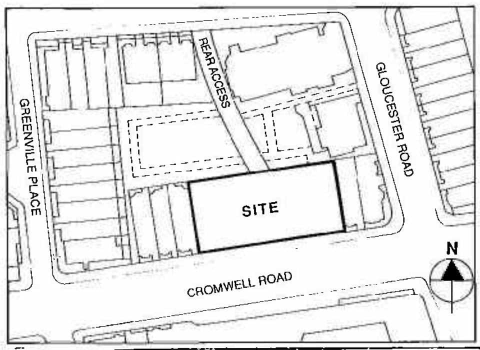


Fig 3.1 b) Location of Buckingham Hotel on Cromwell Road, London

When an engineer commences work on a steel frame design, a thought often passes through his mind 'would there be an advantage in using a rigid frame?' Over the years we have become conditioned to the acceptance of the pin-jointed frame, almost to the total exclusion of rigid frames. This has been encouraged by our design standards, in particular BS 449 'The Use of Structural Steel in Building' which paid scant attention to rigid structures. Matters were not helped by the most successful computer program, FASTRAK, which only accepts the pin-jointed approach to multi-storey designs. Yet our training tells us that if the joints are rigid, the same stress and deflection criteria can be achieved with reduced section sizes, and therefore reduced cost.

The main problem has always been how to achieve effective rigid joints. BS 449 contained an addendum which discussed moment rotation characteristics of standard joints. However, due to the uncertainty of the results and the difficulty of analysis this method was rarely used by industry. In the 1950s the Shell Centre on the South Bank was designed with a fully rigid welded steel frame (Ref 3.1). The cost and complexity of this operation tended to reinforce the view that rigid frames were too expensive for conventional structures. However, with the advent of friction-grip and high-yield bolts, new possibilities in joint connection became available. Ranged against this was an industry of fabricators, computer programmers, designers and estimators who had it firmly fixed in their minds that pinned frames were the only effective solution.

With the replacement of BS 449 by BS 5950 (Ref 3.2) and the imminent introduction of the European Standard EC3, methods of rigid joint designs have moved out of the text books into Building Standards, and gained acceptability – at least as far as the checking authorities are concerned. Although this partial success has been achieved, it is still necessary to prove to designers and fabricators that rigid frame design has both cost and other benefits for the client.

A series of comparative designs made for the Buckingham Hotel project (Ref 3.3) is outlined below to illustrate such benefits, and suggestions are offered for improvement in future rigid frame design options.

The initial scheme

The hotel required the demolition of a row of houses with the retention of their grade II listed facade (Fig 3.1 a, b). Behind this 40m facade a new steel structure would rise approximately 25m above street level. A height limit of 25m was essential, to comply with Section 20 of the London Building Regulations, which would otherwise require a fire-fighting sprinkler system to be provided. To avoid this considerable expenditure every effort had to be made to minimise construction depth. At the same time a better return on investment could be achieved if an additional bedroom floor could be provided. To realise these objectives storey heights and structural zones had to be minimised.

The initial scheme (Fig 3.2) was a pin-jointed steel frame acting compositely with precast concrete floor slabs. Infill concrete between the precast units ensured that the slabs acted as horizontal plates to transfer wind loads to the braced frames in the gable walls some 40m apart at each end of the building. However, the resultant structural depth of 406mm was unfortunately too great. To overcome this, 254mm deep Universal Column (UC) sections were used as beams for the 5m bedroom spans, with the precast floor units recessed into the webs and

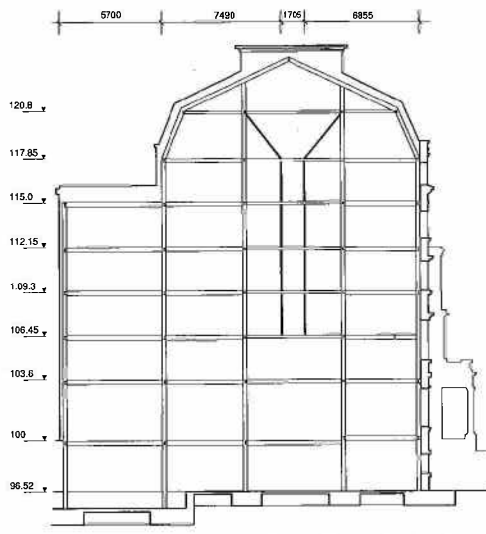


Fig 3.2 Section through initial hotel scheme

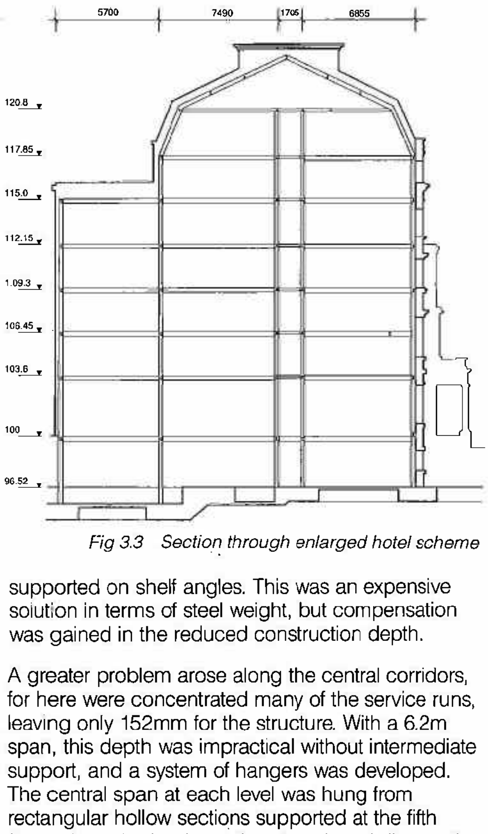


Fig 3.3 Section through enlarged hotel scheme

supported on shelf angles. This was an expensive solution in terms of steel weight, but compensation was gained in the reduced construction depth.

A greater problem arose along the central corridors, for here were concentrated many of the service runs, leaving only 152mm for the structure. With a 6.2m span, this depth was impractical without intermediate support, and a system of hangers was developed. The central span at each level was hung from rectangular hollow sections supported at the fifth floor, where the load was then transferred diagonally across to the main columns, and down to the foundations.

There were good reasons for choosing to hang the floor beams rather than prop them. Primarily, it allowed open planning in the basement and ground floor levels but secondly, a hollow section in tension is smaller than a compression member under the same load and therefore could more easily be concealed within the corridor walls.

The enlarged scheme

Soon after the first scheme was complete, the client purchased two neighbouring properties and the hotel was expanded to give a 56m street frontage. The architect extensively replanned the layout of the public areas at ground level, with a number of major implications for the structure (Fig 3.3).

In order to achieve acceptable column locations in the public areas, bedroom floors were replanned eliminating hangers and giving a more conventional framing. As a result the span of the bedroom units increased from 5m to 7.2m, significantly affecting the beam sizes and therefore increasing the overall height of the development. In addition, the enlarged facade increased the wind load to a level such that if only the braced frames in the gable walls were to be used, the resultant uplift forces could no longer be counteracted by dead load alone. Thus a third braced frame would be needed at an intermediate point. This would have seriously affected the open planning at ground level.

Since neither of these constraints could be accepted by the architect, a significantly different approach was sought, leading to consideration of the rigid frame alternative. This had the advantage of eliminating the crossbracing, thus freeing the planning at ground level, while joint rigidity led to a reduction in beam depths. However, although clearly a simplification, the effects on joint design and fabrication costs had then to be assessed.

Final design

Engineers tend to follow the rule of thumb that minimum material weight equates to minimum cost. Consequently, column sizes for the revised scheme were selected on the basis of compliance with BS 5950 criteria for overall buckling, an approach similar to that adopted for pin-jointed structures. It was appreciated that for some joints the local buckling and bearing capacities of the sections would be exceeded, but calculation showed that with the use of local web plating or horizontal stiffeners, a satisfactory joint detail could be achieved. In this way, the overall weight of steel was kept to a minimum.

The tender documents however, were prepared with a clause giving the steelwork fabricator the option of increasing the column sizes, thus reducing the need for plating or stiffening. In the event the contractor chose to pursue this option, as he was convinced that the extra weight of steel more than offset the cost of designing and fabricating the connections.

For designers, this was a very significant outcome, for it demonstrated the importance of understanding not only the relative efficiencies of pin and rigid design, but also the benefits to be derived from simplicity. As already implied this is not easy to quantify, as simplicity is sometimes derived from increased weight, thus conflicting with the principle of trying to achieve minimum-weight design.

To investigate this further, a re-examination was conducted of the Buckingham Hotel scheme designs, and included an extra model which ignored the tight constraints on construction depth yet maintained the deflection criteria. This additional model gives a useful comparison with earlier scheme designs.

Comparative study of steel weights

With the additional model described above, five possible solutions to the design of the Buckingham Hotel frame were listed:

Design 1 – a pin-jointed frame with UB beams;
Design 2 – a pin-jointed frame with UC beams;
Design 3 – a rigid-joint frame with UC beams, minimum columns and plated connections;
Design 4 – a rigid-joint frame with UC beams, larger columns and simplified connections; and
Design 5 – a rigid-joint frame using UB beams, minimum columns with plated connections.

These five solutions were compared in terms of the weight of steel required (Table 3.1) because the solution adopted by the contractor (Design 4) was relatively heavy. From this comparison it is concluded that reducing the weight, and therefore minimising the material cost, has to be carefully balanced against simplifying the fabrication, which minimises the labour cost. In the light of this finding it is instructive therefore to consider the advantages and disadvantages of each of the five designs.

Table 3.1 Weight of steel per frame for each of the five solutions

		Steel Weight/Frame (Kg)			
	Frame Design Solution	Columns	Beams	Bracing	Total
1	Pinned Frame with UB Beams	4546	7884	1142[1]	13572
2	Pinned Frame with UC Beams	4546	14484	1142[1]	20172
3	Rigid Frame with UC Beams Minimum Columns & Plated/ Stiffened Connections	6716	8115	0	14831
4	Rigid Frame with UC Beams	11338[2]	8115	0	19453
5	Rigid Frame with UB Beams Minimum Columns & Plated/ Stiffened Connections	6716	6388	0	13104

Notes: 1 The weight of the bracing in the end frames has been divided equally between all the frames to give an equivalent weight per frame.
2 Grade 50 steel used in the columns.

Design 1 adopts a pin-jointed frame with UB beams. In order to use this design, the storey height would need to be increased to provide adequate service space and ceiling heights. This would increase the overall height of the building leading to additional costs for sprinklers and cladding. Clearly, the cheapest structural solution is not always the most economic overall.

To reduce the overall structural depth, Design 2 used UC sections for the beams, but with the same structural layout as Design 1. The use of UCs instead of UBs increases the weight of the beams by 84%, and the weight of the total steelwork by 49%. Although apparently uneconomic, the extra cost of the steelwork can be more than justified by the savings made elsewhere.

Design 3 is derived from schemes for the enlarged hotel and has a rigid frame using the smallest possible columns. The column weight is greater than in the pinned frame solution due to the greater bending moments induced through the rigid connections. However, comparison with Design 2 shows a considerable reduction in the weight of UC beams because the support moments significantly reduce the mid-span moment and deflection, particularly for the long-span bedroom floor beams. It is also interesting to note that UC beams in the rigid-frame solution, Design 3, are only slightly heavier than UB beams in the pinned-frame solution, Design 1. It would however, be misleading to assume that the weight benefits derived in Design 3 will be entirely realised in the resulting costs per frame. As previously described, minimal columns require plating and stiffening in order to improve the local buckling and bearing capacities of the connections. This extra fabrication cost is not reflected in Table 3.1, thus Design 3 is not as economic as it might at first appear.

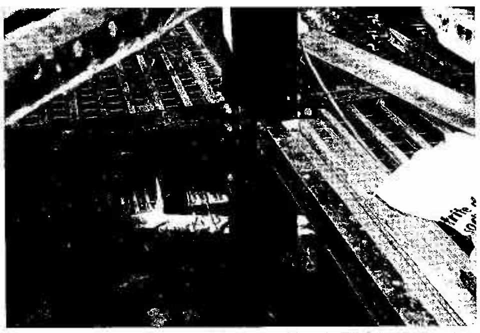


Fig 3.4 Buckingham Hotel: Rigid connection

Design 4 provides some insight into the cost consequences of complex rigid connections. It is normal practice for connections to be quantified as a percentage allowance in the Bill of Quantities, which for simple pinned connections is usually taken as about 10% of the frame weight. As described earlier,

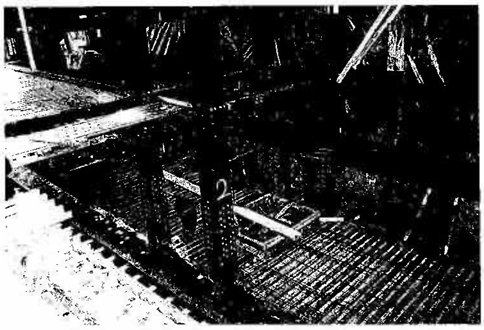


Fig 3.5 Buckingham Hotel: Portal frame connection showing column splice

the contractor on the Buckingham Hotel project chose the option of increasing the column size instead of plating and stiffening the columns to achieve rigid connections (Figs 3.4, 3.5). If the contractor's commercial decision is considered sound, then it can be said that the cost of Design 3 (14831 Kg of steelwork with complex rigid connections) is equivalent to or greater than that of Design 4 (19453 Kg of steel, with simple rigid connections and columns in grade 50 steel). Design 4 is 31% heavier than Design 3 and one can deduce that the cost of stiffening a joint to make it a rigid connection is 30–40% as expressed in steel weight, compared to 10% in providing a pin-joint connection.

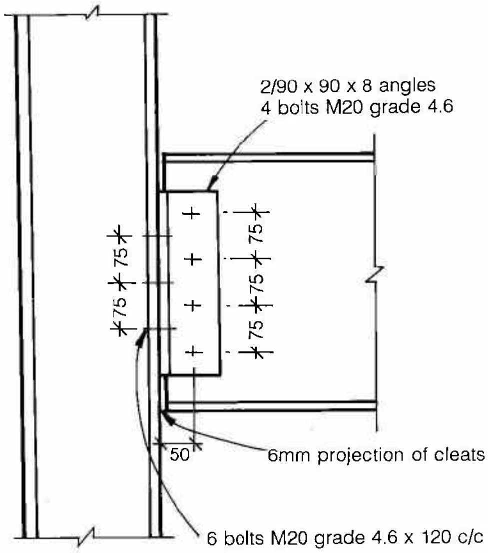


Fig 3.6 Typical pinned beam to column connection

Design 5 is a rigid frame using UB beams. This is likely to be the more conventional case and apparently produces the scheme with the most economic steel weight. However, as mentioned

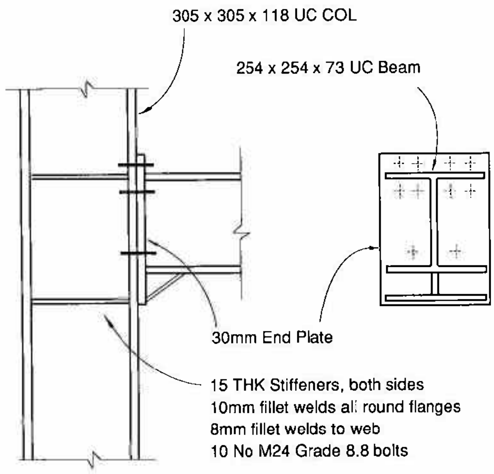
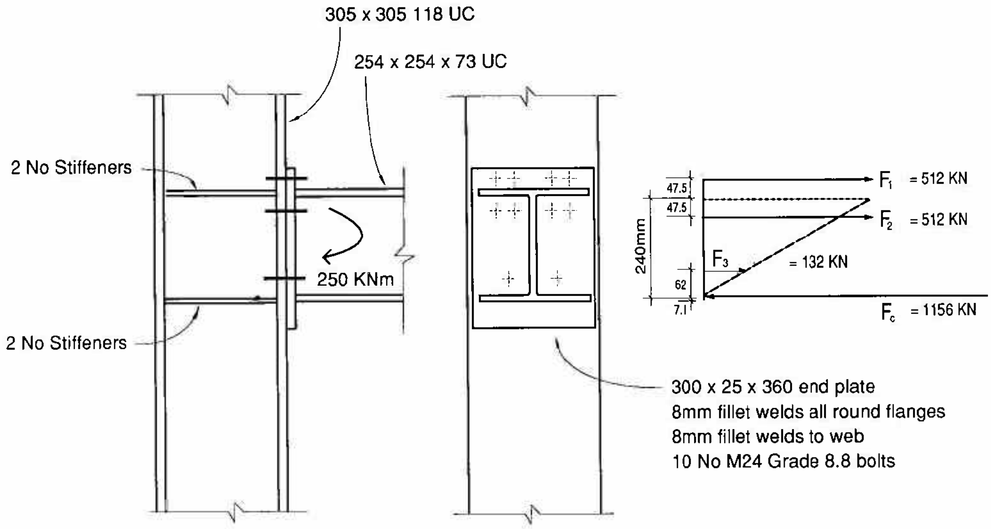


Fig 3.7 Typical rigid connection

above, consideration must be given to the cost of joint design. This design could not be adopted as it still infringed the 25m height limit.

Joint design

In order to achieve the benefits that rigid frame design can bring, it is instructive to consider joint design in a little more detail. Figures 3.6 and 3.7 show typical pinned and rigid connections (Ref 3.4). From these details, it can be seen that there are indeed extra costs in fabricating rigid connections. A word of warning should be added at this point. The rigid details adopted were complicated by the use of UCs as beams. The shallow depth of these sections leaves only a small lever arm to carry bending moments, resulting in high push/pull reactions. Practical considerations dictate that the tensile force should never have to be resisted by more than four bolts, otherwise extensive plating and stiffening is required to ensure that the load is shared equally between all bolts (Ref 3.5). Further, where depth permits, it is advantageous to introduce haunches to increase the lever arm so reducing bolt forces and the need for plating. Of course this is the fundamental reason why UB sections are used for beams in preference to UCs, as their increased depth reduces the tension and compression forces in the flanges. At a rigid end connection, these forces have to be transmitted to the columns, so again keeping these forces low reduces the stiffening required (Fig 3.8 a, b). However, it is still likely that connection detailing is going to be more costly than the 10% allowance made for pin-jointed structures.



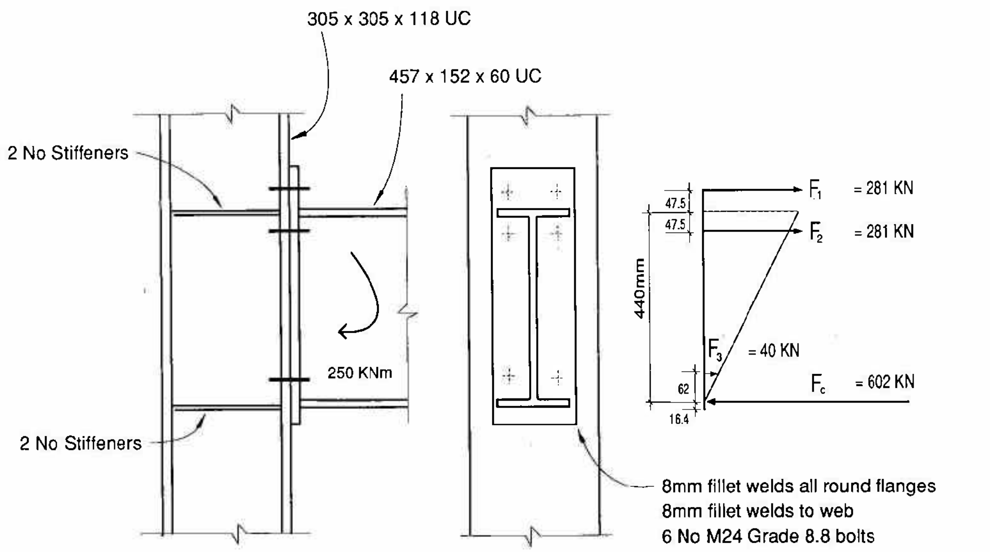


Fig 3.8 a) b) Beam to column connection – bolted end plate

Overall conclusions are two-fold. Firstly, although conventional pinned frames are the most economic conclusion, rigid frames are a valuable option for medium height structures if there is sufficient structural depth at each floor to allow use of UB beams or haunches of their equivalent depth. Rigid frames have the advantage of allowing the architect to plan his layout without the need to consider restrictions imposed by bracing.

Secondly, if UC sections have to be used as beams, then it should be stressed at an early stage that the structure will be expensive. It is important to appreciate that UC sections are not intended for use as beams, and it may be advisable where UC beams have to be used to consider a pinned frame solution with bracing. However, with the growing complexity of modern buildings, it is becoming common for many factors other than steel weight alone to control the economics of the chosen design.

John Morrison and Hugh Wildy

References

3.1 Measor E O and Williams G J: 'Features in the design and construction of the Shell Centre, London' Journal of the Institution of Civil Engineers, March 1962

3.2 British Standards Institution BS 5950 : Part 1 : 1985 'Structural use of steelwork in buildings' 1985.

3.3 Brooke P: 'Buckingham Hotel – Renovation in Kensington, London' Patterns 4 pp 23–28 Nov 88.

3.4 Owens G W and Cheal B D: 'Structural steelwork connections' Butterworths

3.5 Packer J A and Morris L J: 'A limit state design method for the tension region of bolted beam-column connections' Journal of Institution of Structural Engineers, October 1977

A British School in Armenia, USSR

Project data

Client	Department of Education and Science
Architect	Department of Education and Science, in conjunction with Building Design Partnership
Structural Engineers	Buro Happold
Services Engineers	Buro Happold
Quantity Surveyors	Department of Education and Science
Management Contractor	Mowlems International
Project Value	Not available
Completion Date	August 1990

Fig 4.1 Post earthquake devastation in Armenia (Credit: John Walmsley)

On the morning of 7 December 1988, a devastating earthquake measuring 6.9 on the Richter scale struck the Soviet Republic of Armenia. In a series of tremors lasting on average 45 seconds, the small town of Spitak was virtually levelled to the ground, whilst in Leninakan, the second largest city in the republic, 70% of all buildings collapsed or were rendered unsafe. Few multistorey buildings remained intact and of the 228,000 inhabitants of Leninakan almost all were made homeless (Fig 4.1).

Although early reports of over 200,000 casualties proved exaggerated, the final total of deaths reached approximately 30,000, itself an appalling loss of life. At least 15,000 people died in Leninakan alone. One of the sad facts of the earthquake was its timing, 11.41am, when all the children were in school. As a result a very high proportion of the dead were children and teachers – all but 6 of Leninakan's 37 schools were destroyed.

At the time of the earthquake the Soviet Union's President Gorbachev was in the United States on the first leg of a journey which would have brought him to London a few days later. As soon as he was informed of the disaster all engagements were cancelled and he returned to the Soviet Union. Prime Minister Margaret Thatcher expressed her full understanding and sympathy and announced that Britain would be ready to help in the relief exercise.

On 9 December 1988 the UK Minister for Overseas Development stated that £5 million would be made available for Armenian relief and reconstruction. Of this £5m, £1m was spent during the initial emergency operations with the balance thus available for longer term construction.

The inception of the British school

The proposal to offer Armenia a replacement school as a gift was made by the British Government in early January 1989. The idea was warmly received by the Soviets who explained the minimum size of schools within their educational system would normally accommodate 400 pupils aged between 6 and 16 years. The proposed school would specialise in the teaching of the English language and it would replace a similar, although larger school destroyed in the earthquake. The project was to be designed and managed by the Department of Education (DES).

A delegation led by John Wiggins, Deputy Secretary of the DES, visited Moscow and Armenia during late January. It met with senior representatives of the USSR Government in Moscow and, in Armenia, local administrators and representatives of the Armenian Church. Discussions further underlined the need for the school and established Leninakan as its location (Fig 4.2).

At the beginning of February 1989 the DES Architects and Building Branch were given the go ahead to proceed with the scheme as fast as possible. The DES was to be directly responsible for the brief, sketch design and site supervision together with the overall management of the construction process. The Building Design Partnership (BDP) was appointed to produce the detail design and cost advice while Buro Happold

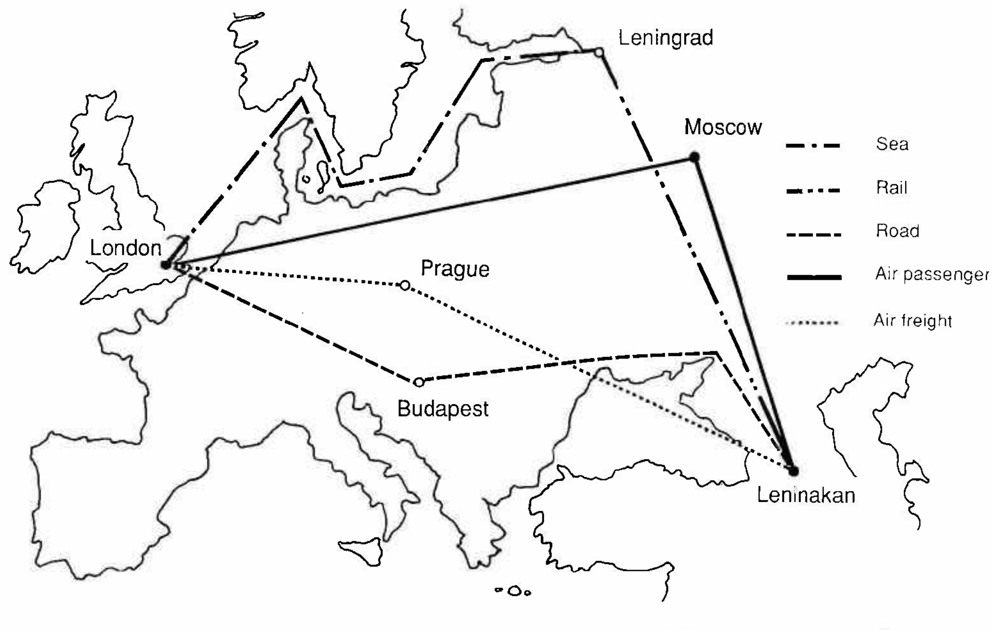


Fig 4.2 Location and communication routes of Leninakan within Soviet Republic of Armenia

were appointed as structural and building services consultants. The design team was complemented by the early appointment of the management contractor Mowlem International Ltd (MIL), which had extensive overseas experience, having worked in the Falkland Islands.

Sketch design

In parallel with design studies by the DES, Buro Happold were asked to consider the implications of building form in view of the area's seismicity. Conclusions favoured a single storey construction built as a series of pavilions, linked by glazed walkways. Movement joints would be located in the connection of walkways and pavilions, so avoiding the need for 30m spacing of joints through a single larger structure. The schedule of accommodation resulting from the DES studies lent itself to this approach with a total of six pavilions being required for administration, languages, science and maths, arts, crafts and music, physical education and infants (Fig 4.3).

A delegation led by the DES Chief Architect, Peter Benwell made a second visit to Armenia in March 1989. The main objectives of the visit were to approve the sketch design, to agree on the site and to approve the programme of construction and practical arrangements. With a representative on this second visit, Buro Happold had a separate set of objectives – to ascertain weather data/design conditions, to confirm utilities data, to view local standards of construction and servicing, to confirm design data, including lighting levels and internal temperatures, and to establish local practices and methods on maintenance and operations of building systems.

All objectives of the delegation were achieved and embodied in a protocol document which clearly defined responsibilities agreed between the Armenians and the UK authorities. The UK agreement stated that the school would be completed by 31 August 1990. President Gorbachev's visit to London on 6 April 1989 provided the opportunity to formalise the status of the project by the signing of a Memorandum of Understanding which validated the protocol, and work was able to commence on site in May 1989.

Criteria for earthquake design

Leninakan is the second largest city of Armenia with a population of 200,000. It is 1300m above sea level in the Caucasus region of Alpine period folded mountains thus lying in a zone of extreme instability and constant neo-tectonic movement. It is one of the few cities of the world unfortunate enough to have been devastated by earthquake twice in one century, the last major quake occurring in 1926. Furthermore, during the first half of the 20th century

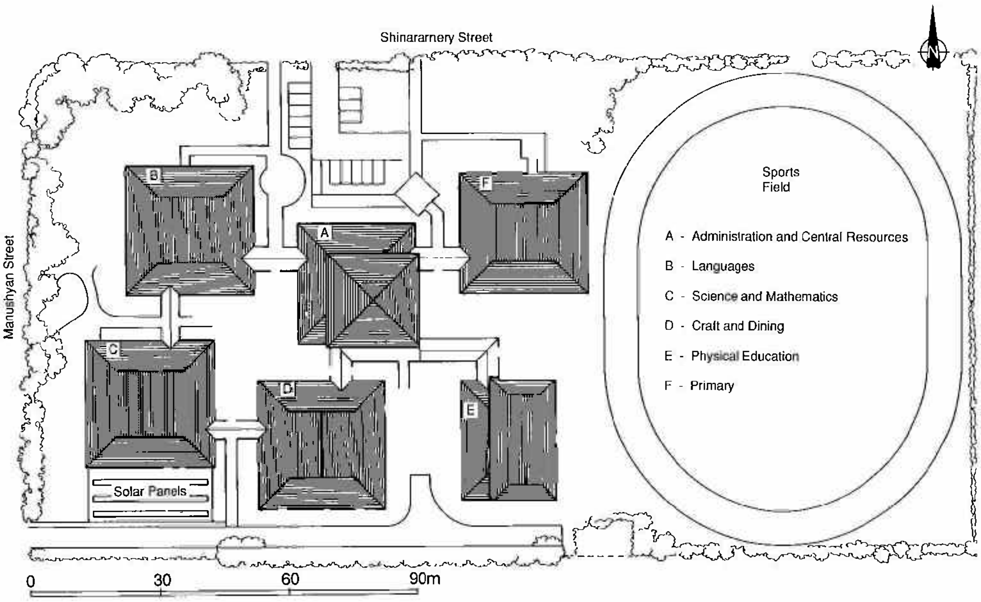


Fig 4.3 Layout plan for six pavilions of British Schoo

there were an average of two quakes per year in the area – over one hundred earthquakes in 50 years, although most of these were relatively minor.

The new school building was therefore required to be earthquake resistant. This was interpreted as being capable of sustaining the design earthquake with a minimum amount of superficial damage so enabling the school to remain operational – a requirement corresponding to Category 1 of the 1970 Russian Code (Ref 4.1) and to Category 2a of the New Zealand Code (Ref 4.2).

Earthquake design of building structures is chiefly involved with resisting mainly horizontal inertia forces generated by ground movements. The shock produces a fluctuating signal of acceleration, velocity and displacements in the parent rock. If the site is underlain by a depth of soft alluvial soil, this can have its own natural frequency, which will reduce the effect of other frequencies while enhancing movements at its natural frequency. The building will also have a response which will contain a number of natural frequencies. These frequencies can be found by a dynamic modal analysis which predicts the mode shapes and their natural frequencies. If there is sufficient energy in the earthquake ground movements at these frequencies, then deflections of the structure and consequently the internal forces will be magnified.

The purely elastic response is modified by damping, which can come from a variety of sources including the inelastic behaviour of the structure, friction in the cladding and finishes and energy absorption within the subsoil. However, the level of damping generally increases with the level of vibration, this applying particularly to damping caused by ductile behaviour of the structure. A full analysis with modelling of ductile behaviour can be carried out but this requires a time history analysis of the whole structure to yielding of the steel members, and the adoption of further assumptions.

Building code design methods involve designing the structure for nominal horizontal inertia forces, which are defined by the seismicity of the area with modifications for the response and ductility of the structure. The Russian Code follows this procedure and was used for the basic design of the structure, giving an inertia force coefficient of 0.3g. The forces derived were also checked against the New Zealand Code, producing a similar coefficient.

Dynamic analysis and ductility demand

As a check on the code design methods, a dynamic modal analysis was carried out by Dr Colin Taylor of Bristol University for Blocks B and E of the school complex. In the analysis, a conservative response spectrum of .75g at all frequencies was used. This was taken from the response spectra for the Spitak earthquake as published in the ESEE/EFTU Research Report No 89/1 (Ref 4.3) with the response modified for a damping coefficient of 2%. This measurement was taken on a rock site at Gukasyan, 30km from the epicentre. The site at Leninakan is on alluvial deposits overlying tuff bed rock, although the exact depth of the alluvium is not known. This alluvium will have the effect of reducing the high frequency accelerations and increasing the amplitude at lower frequencies.

The ductility demand is defined in the Bristol University work as the ratio of the calculated elastic deflection to the deflection at yield. The maximum calculated deflection at the top of the columns is 9.5mm while the deflection at the yield of the ductile connection would be 4.4mm. Hence the amount of ductility required is 5.1mm horizontally or 3.6mm in the direction of the diagonal. The ductile connection has a bendable plate with 60mm clear length between the face of the diagonal member and the space plates. The deflection of 3.6mm is small in comparison to this and can easily be accommodated.

Adaptation of the SCOLA system

From initial conception the school was planned to be built using the SCOLA system. This British modular system is primarily a structural steel frame based on 150mm × 150mm square hollow section columns positioned at the intersection of grid points. A range of standard beam units can be used to build suspended floors, flat and pitched roofs. With the grid based on multiples of 1.8m, the preferred minimum dimension is 3.6m, with roof slopes at a gradient of 1 in 3.

Buro Happold were originally asked to verify the earthquake resistance of the SCOLA system but it soon became apparent that a number of details would have to be modified and additional components added. Apart from essential bracing, these were mostly secondary structural elements required for supporting walls and ceilings.

Fig 4.4 Typical "square" block showing link to adjoining pavilion

Early on in the design process it was decided to construct the school as a number of square or nearly square blocks with a maximum dimension of 30m (Fig 4.4). These square blocks, with bracing around the perimeter walls, are an ideal form for resisting earthquake forces. Blocks would be joined by links which could also act as entrance lobbies. The advantages of this arrangement were that the blocks could be separated from each other by generous movement joints which would allow for the effects of thermal expansion and earthquake movement. Furthermore, with this size of block, the pitched roofs could drain to the perimeter without the need for internal valley gutters.

In addition to earthquake loadings, the buildings were designed to resist both local snow and wind loads. The ground snow load of 1 kN/m^2, derived from the best available information, was more severe than the typical UK snow loads normally required of the SCOLA system and resulted in increases in the section sizes of roof rafters and purlins. The forces from wind loads were generally less than those from either snow or earthquake.

Resistance to horizontal forces can be provided either by beams and columns acting together as portalised frames or by including diagonal bracing between beams and columns. An initial check was made on the value of the portalised frames and it was found that the members would have to be considerably increased and the standard SCOLA connection altered. As deflections would be large, causing greater damage to the external and internal walls, it was decided to opt for the braced frame system with two braced bays on each side of the blocks and a tie beam at eaves level to distribute the horizontal forces to the braced bays (Fig 4.5).

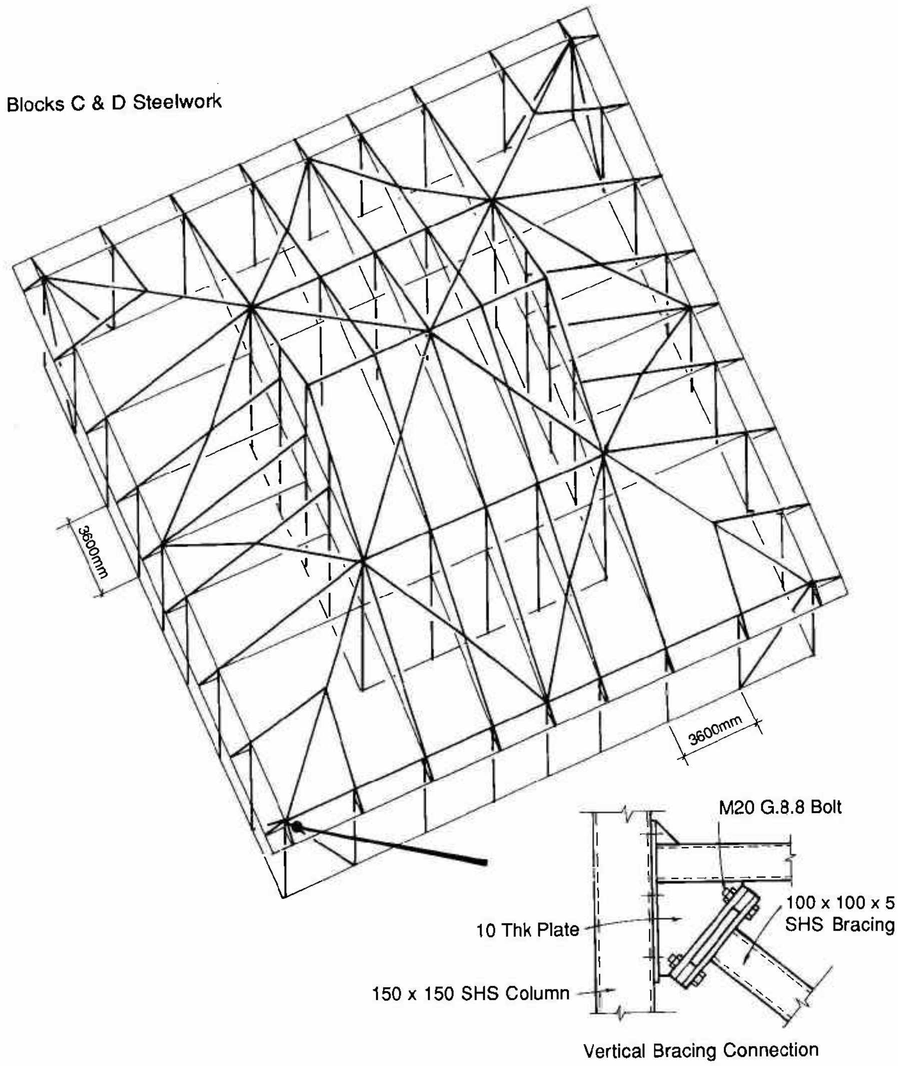


Fig 4.5 Steel bracing of a typical block

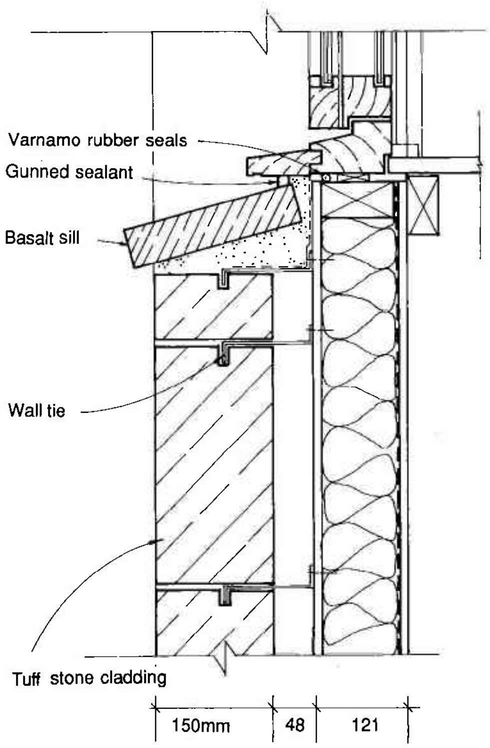


Fig 4.6 Detail of fixings in stone walling

These horizontal forces must also be transmitted from the roof to the side walls. Following debate on the ability of the TAC trays and purlins of the roofing system to provide the necessary shear plate action, it was decided that it would be necessary to introduce a significant number of special fixings and strengthened members which, being outside the SCOLA system, were difficult to control in Armenia. Accordingly a bracing system was introduced into the roof as an alternative solution.

One of the prime considerations in earthquake design is that the members and particularly the connections, should fail in a ductile mode. Although this is generally the case with the SCOLA details, an additional connection with designed ductility in the diagonal braces in the walls was introduced. This consists of a cross plate at the top end of the braces which will yield in bending at the design load, so providing a huge amount of damping.

As a result of the cross braced bays, horizontal forces generate uplift in the edge beams. These therefore require sufficient mass and bending stiffness to resist such forces. Because of the nature of the ground and the depth of frost penetration, it has not been necessary to significantly increase the size of these beams. In fact, there is generally an additional 500mm of mass concrete below the ground beams.

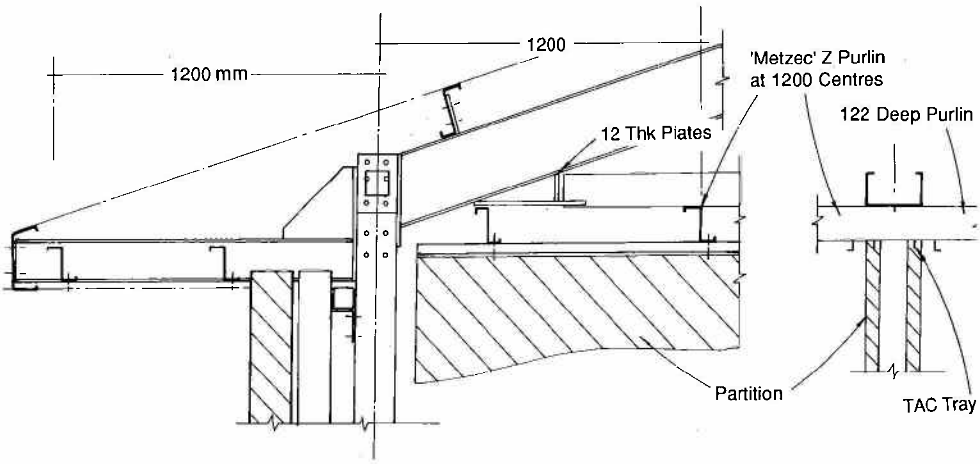


Fig 4.7 TAC tray panels and Z purlins of roof construction

Secondary structural elements

Apart from ensuring that the main frame is capable of resisting the earthquake forces, it is necessary to ascertain that the building elements, walls, ceilings and roofing remain attached to the frame so maintaining their integrity. Nominal inertia force coefficients of these elements are not well defined in the Russian Code, at just greater than 0.2g. In the New Zealand Code a schedule gives a range of forces for different situations. For single storey buildings, the maximum acceleration is taken as 0.6g with nominal acceleration of 0.6g for ceilings and 0.3g for walls. Consequently a blanket coefficient of 0.5g for all components was selected for the project.

The external walling consists of 150mm tuff blocks of mass 300Kg/m^2, which with a coefficient of 0.5g, have an inertia force of 1.5kN/m^2. This is similar to local wind pressures on tall buildings in high wind speed areas, hence a similar fixing treatment was considered adequate. In the case of the tuff walls, the ties must be securely anchored to the stone work and a satisfactory fixing path must be taken back to the primary steel work. Steel wall cladding rails at eaves level and at window sill level have been included for this purpose. Fixings to the stone walling, in the form of wall ties secured by dowel to the stone and screwed back to the timber studs in the lining wall were inserted in every course. These fixings should be capable of taking a nominal load of 1.0kN (Fig 4.6).

The sloping roofs produce heights of 6m to 8m in the centre of the blocks. In the perimeter teaching rooms a ceiling height of 3m was chosen, level with the tops of the Stelvelite partition walls. Both the partition walls and the ceilings had to be securely fixed against horizontal inertia forces. The normal method of fixing these elements by hanging rods and wires to the roof structure, here up to 3m above, was completely inadequate in this case for earthquake conditions. To provide satisfactory fixings a ceiling grid of cold formed channels and Z purlins was introduced. This grid also provides support for service pipes and cable trays and has been designed for access loads. Inertia force coefficients of 0.5g were used for the design of the ceiling grid members, but were later amended for service pipe fixings.

The roof construction consists of TAC tray panels on Z purlins (Fig 4.7). Trays are filled with insulation and closed with felt battens and artificial 1.5Kg 300 × 260mm slates (mass = 1.5kg) secured by nails into the battens. Slates are fixed with two nails at the sides and a wire tag at the bottom. Fixings would in some locations be required to take wind forces of 1kN/m^2 (7.8Kg/slate), well in excess of the inertia force. Normal fixings of the TAC trays to the Z purlins with three screws at each purlin are also adequate.

Building services

Following discussions with the Armenian authorities an early decision was taken that all services would be designed to British Standards. This was accepted by the Armenian and Soviet representatives, which simplified the design process and allowed early procurement of materials by Ellis Mechanical Services, the appointed M&E sub-contractors.

The Armenian climate is characterised by very warm summers (average external temperature +32°C) and extremely cold winters (winter external design temperature −23°C), with a consequent yearly range of approximately 50°C. As the winter daily temperature rarely rises above freezing, permafrost penetrates the ground up to a depth of

0.5m. The summers are hot and dusty, interjected with periods of intense rainfall which turn the ground clay into an unmanageable slurry. The accompanying lightning storms are a dramatic sight.

With such extremes of temperature the first thermal objective was to insulate the buildings to a high standard and to use good quality timber frame double glazed window units. The following table lists the 'U' valves that were achieved within the construction and financial cost limits of the project.

Table 4.1 'U' values of building elements

Element	U Value
Walls	0.25 W/m^2°C
Roof	0.25 W/m^2°C
Floor	0.60 W/m^2°C
Windows	3.2 W/m^2°C

The town of Leninakan is served with a district heating scheme, the underground mains of which were still in operation despite the earthquake. This system, operating with a flow temperature of 90°C and a return temperature of 60°C, provided an adequate source of heat for the school.

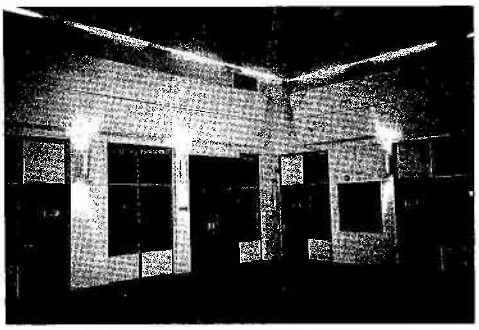


Fig 4.8 Internal courtyard within teaching pavilion

With a total heat demand of 1000kW (for 4000m^2) heat distribution around the school is at 85°C flow and 80°C return. Primary distribution is run out as a constant temperature circuit from the schools heat exchanger in Block E. Each block then takes its heat demand into six individual block pumping stations. Here a weather compensated circuit is provided for radiators and a constant temperature circuit for radiant panels and air curtain heaters.

The links between the blocks are unheated, providing sheltered circulation between the buildings, and effective draught lobbies for the block entrances. Each block has a central resource area, 'an internal courtyard' into which the children can retreat when the weather prevents play outside (Fig 4.8). As this area was to be as much a retreat in summer as in winter, the space was given volume to allow effective natural ventilation, removing heat and moisture build up at the apex of the building (Fig 4.9). In winter the central space is heated by radiant panels mounted at 3m above floor level. This method of heating was selected because it clears the floor making the area safe for children to play, and does not produce unwanted fan noise. Radiant heat was also considered to be more appropriate with lower air temperture for the same environmental temperature (Fig 4.10).

The surrounding classrooms are 7m deep and are naturally ventilated. To encourage ventilation across the room, grilles in the ceiling along the inner wall take air to the high point of the central resource for discharge to outside (Fig 4.9). Mechanical ventilation is confined to specific areas, dining hall and kitchen, sports hall, changing room and internal toilets.

The ceiling voids above the classroom provided an excellent space for the distribution of services within each block. Here pipes and cable trays are fixed to the secondary steelwork with space still available for maintenance staff. Primary services between blocks are run in the ceiling space of the unheated link block and are trace heated to provide protection from the extreme cold (Fig 4.11).

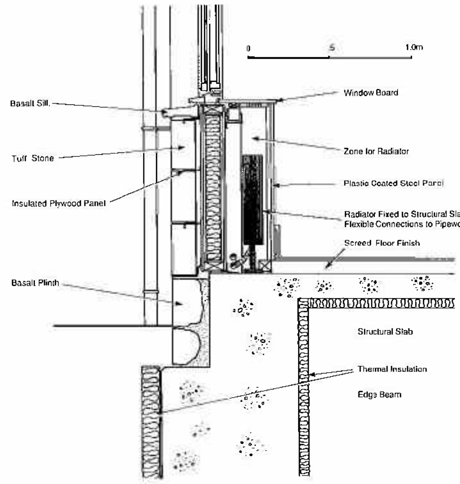


Fig 4.10 Section through perimeter heating showing insulation and connections to structural slab

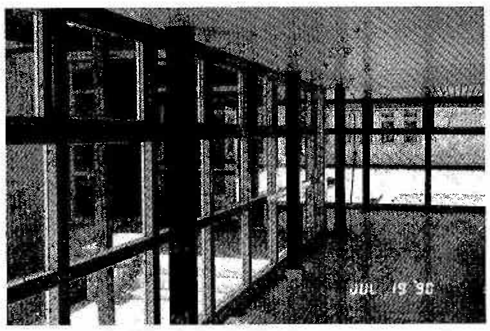


Fig 4.11 Unheated link block between pavilions

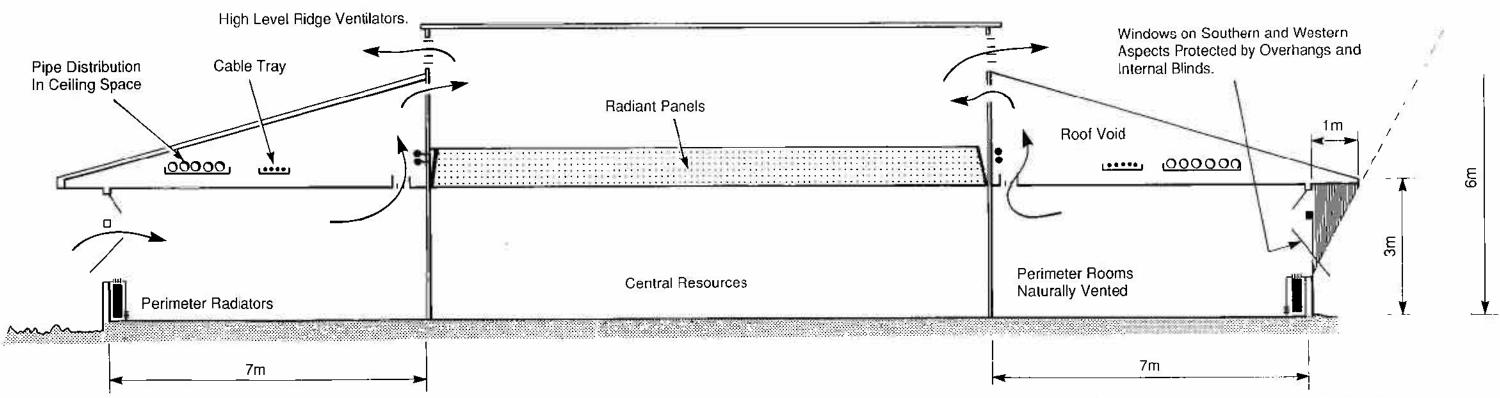


Fig 4.9 Environmental section through a typical block

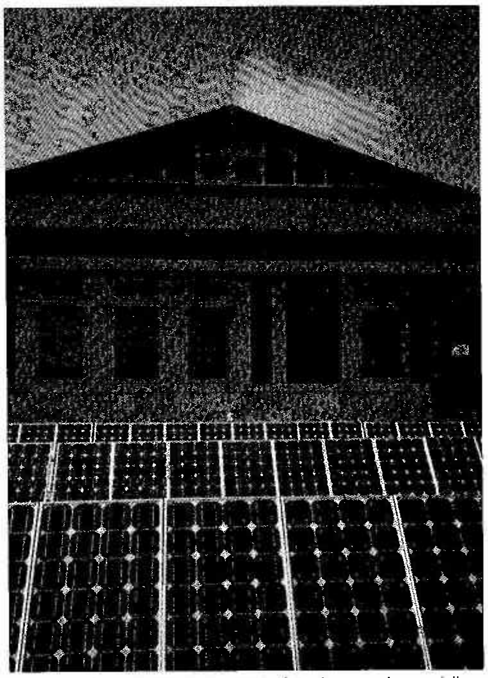


Fig 4.12 Array of photovoltaic solar panels providing power to computer science laboratories

The construction of a new sub-station on the site by the Armenians provided the school with a three phase 380V supply line rated at 500kVA into the primary switch and metering room in Block C. Electrical distribution is provided from the main panel in Block C with three phase radial circuits run out to a single distribution board for each block. The only exception is that an additional radial circuit is provided for the electric kitchen. Within each block electrical services are standard but incorporate a photovoltaic solar panel system donated by BP Solar (Fig 4.12). An array of panels situated to the south of Block C provide light and power to one of the computer science laboratories in this block. The system was selected and installed with an envisaged additional use as a teaching tool. A panel in the resource area of Block C displays the circuits and monitors conditions and outputs of the system at any instant.

Seismic design of services

The fact that the school is single storey and the adoption of the pavilion scheme greatly simplified the seismic precautions that had to be taken in services design. There was however a great deal of consultation between both structural and services engineers to ensure that structural components to which services were attached could withstand the dynamic forces imposed, and to achieve a similar standard of confidence for services design consistent with that adopted for the rest of the building. If the school was to receive minor structural damage in an earthquake but could be made habitable, then the services installation would be expected to be in no worse condition. If however, the school was to be made uninhabitable for a prolonged period then the condition of the services would be unimportant – providing of course that the services failure had not caused secondary damage or casualties.

Britain has no design standard or code of practice for earthquake design and so it was decided that the New Zealand Code (Ref 4.2) would be used as the design standard for provision of mechanical and electrical services. As a result, within each structurally independent block and in the links, services fixings are rigid with a design coefficient of 2.0g. Services connections between link and block are provided with flexible connections and offsets in distribution lines.

Following the commencement of construction on site in May 1989, work proceeded smoothly, rapidly achieving completion in August 1990. The completed building (Fig 4.13 a, b) named the Lord Byron School, was officially opened on 10 June 1990 by the Prime Minister, Margaret Thatcher.

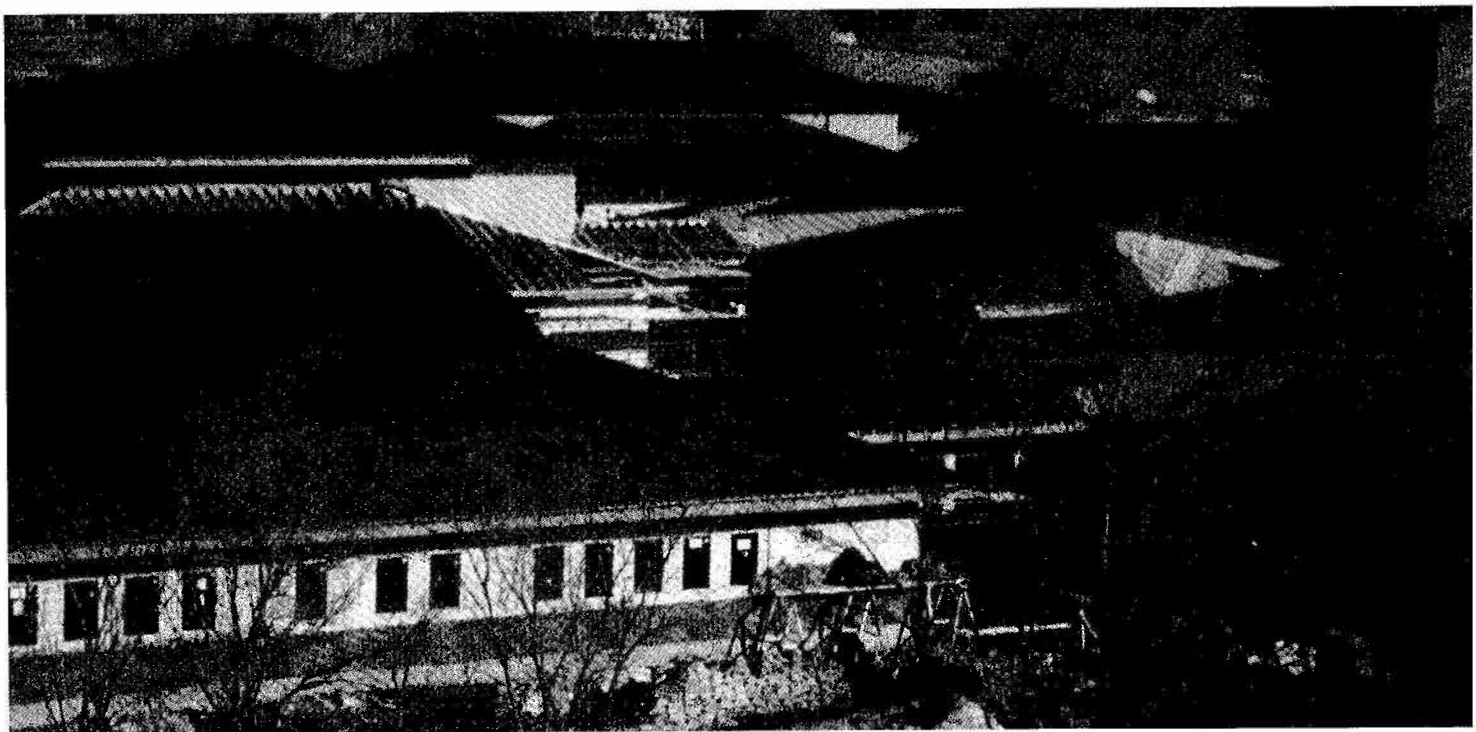
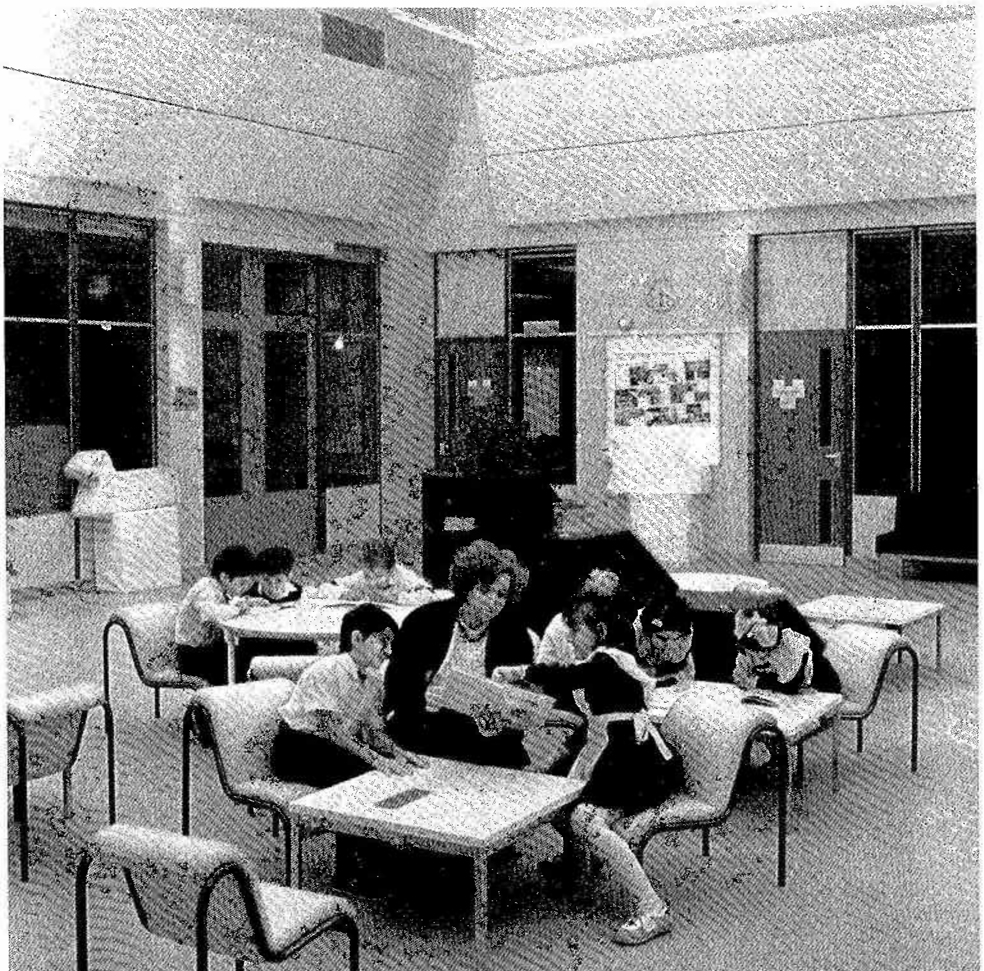


Fig 4.13 a) Exterior of completed sch



b) Interior view of school in use (Credit: John Walmsley)

Facilities were handed over to the Soviet authorities on 1 August 1990 and the school opened for classes on 1 September 1990 (Fig 4.14).

This has been a rewarding project on which to work, with a heartening team spirit from everyone involved. To take a project from conception to completion in 19 months is indeed commendable, more so when one considers difficulties of location and transportation of all labour and materials from the United Kingdom.

Tony McLaughlin and Ian Liddell

(With thanks to Jeremy Wilson and Liz Lloyd Jones of the Department of Education and Science.)

References

4.1 SNIP II – 6 (USSR 10:07) State Committee for building of the Soviet Ministers of the USSR. Part II Section A Chapter II 'Loads and Effects – Design Standards' 1962 + 1976 (draft).
4.2 NZS 4219: 1983 New Zealand Standard 'Specification for seismic resistance of engineering systems in buildings' 1983.
4.3 ESEE/EFTU Reasearch Report No 89/1.

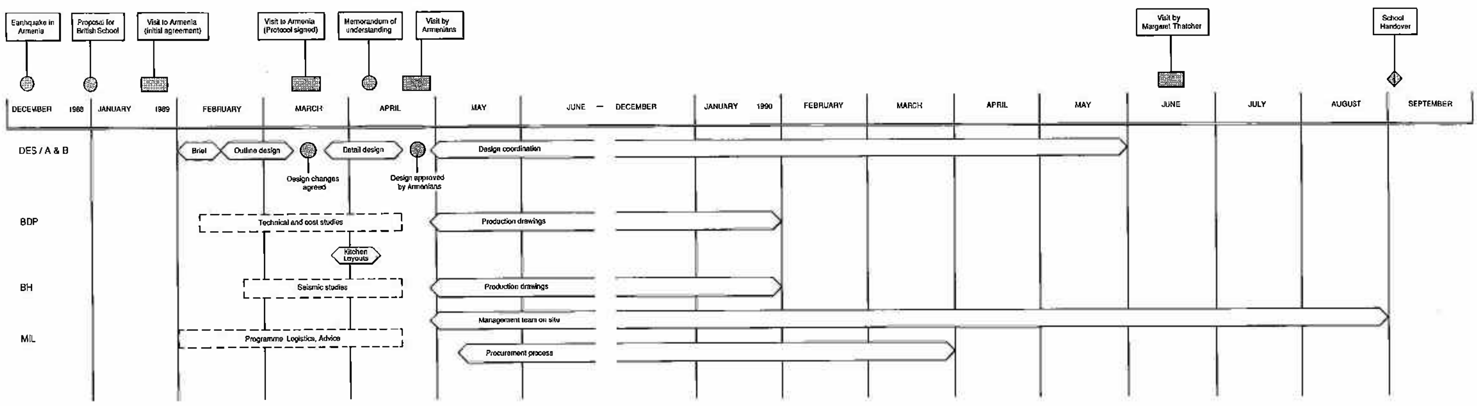


Fig 4.14 Programme of project from earthquake in December 1988 to completion of school August 1990

Hooke Park College, Dorset – house and training centre in green roundwood

Project data

Client	John Makepeace, Parnham Trust
Architect	Ahrends, Burton & Koralek with Frei Otto
Structural Engineers	Buro Happold
Services Engineers	Buro Happold
Quantity Surveyors	Bernard Williams Assoc.
Contractors	Ernest Ireland Const. Ltd & Hooke Park Const. Ltd
Project Value	£450,000
Completion Date	Prototype House: 1986 Training Workshop: Autumn 1989

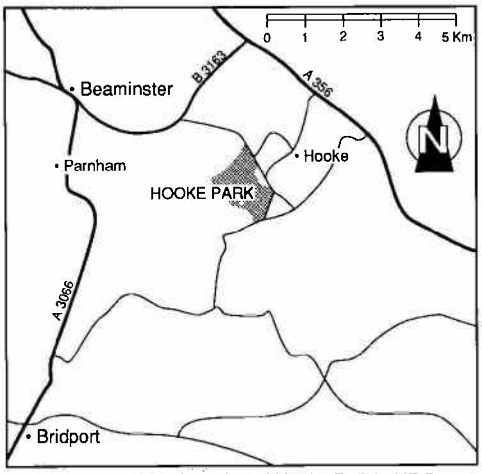


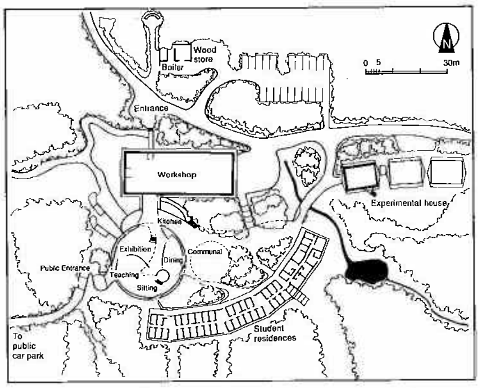
Fig 5.1 Location of Hooke Park in NE Dorset

'Founded in 1977 as a non profit making educational charity, the Parnham Trust is concerned with the teaching and development of the design, production and business skills needed for the establishment of new enterprises using timber as their primary material'. – So states the guidebook to Parnham House, home of John Makepeace who has masterminded the concept of Hooke Park as a training centre in the use of green roundwood, deep in the heart of a Dorset woodland. Roundwood has been recognised as a viable construction material worldwide and throughout history. However, it is the use of immature thinnings of little commercial value that is innovative at Hooke. Indeed, the college buildings themselves are a remarkable manifestation of this low cost, renewable construction material.

Lying in north east Dorset (Fig 5.1), Hooke Forest was typical of many neglected UK woodlands with its poor quality mature trees. The Parnham Trust intends to show that thinnings from such a forest can have commercial value, which will in turn allow resources to become available to work the woodland properly, further permitting the production of a better quality mature timber. Greenwood timber, available as 7m poles of 50mm to 100mm diameter and as 10–15m poles of 200mm base diameter, was to be used for the family of three types of building proposed for the college site. The first, a prototype house, is now complete (Fig 5.2 a), temporarily housing a seminar room and administration offices. The second, a training centre highly commended by judges in the 1990 British Construction Industry Awards (Fig 5.2b), has just reached completion and the third, yet to commence, will accommodate a visitor centre.

Development of new technology

Throughout the project roundwood has been used in two particular ways – as spars in bending, tension or compression to form rigid frames, floors, platforms and bridges, and as flexible poles in roofs. Previous use of tension members in timber buildings has been limited by inefficient traditional joints. Alternative jointing methods were therefore investigated and tested in the scheme for the prototype house. A joint was developed consisting of a steel rod embedded in epoxy resin within a stepped drilled hole (Fig 5.3). The stepped hole allows the ends of the hollow wood fibres to be exposed to give improved penetration by low viscosity two part epoxy resin glue with added cellulose microfibre filler. The filler improves the gap filling properties and increases the elasticity of the joint.



b) Original site layout showing workshop and experimental house

A compression joint, very similar to the tension joint, was also developed. In this joint, the cross grain strength of the timber is increased as the fibres are filled locally and strengthened against crushing.

The concept in the roof shell of the workshop was to use the wood in a prestressed condition as flexible poles using ultimate permissible stresses. Such a method must ensure that reduction of the prestress does not lead to structural instability, that applied loads can be safely carried at the ultimate

Fig 5.2 a) External elevation of completed prototype house

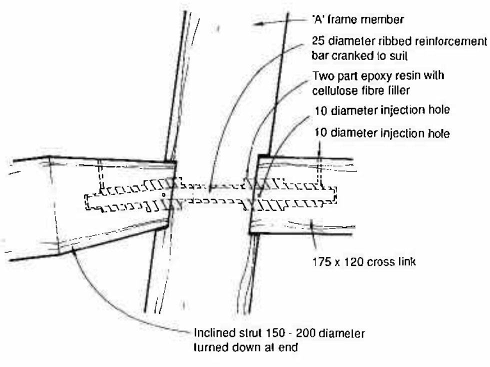


Fig 5.3 Detail of steel rod joint in stepped drilled hole

limit state and that deflections under short term loads satisfy serviceability requirements.

The method used to induce prestress involved bending the timber in its green state. Stresses induced in this process can dominate the performance of the structure and so laboratory tests were carried out to see how quickly this stress would be relaxed by the visco-elastic strain of the wood, before construction began (Fig 5.4).

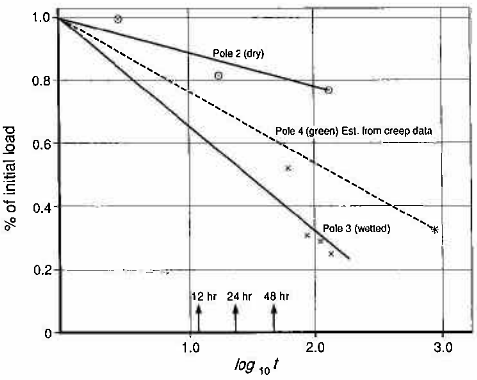


Fig 5.4 Results of trial loading of poles

Site investigations and foundations

The site is located on a slope below a steep escarpment where major landslides occurred at the end of the last glacial period and minor slides have followed over the past few thousand years (Fig 5.5). Although no evidence of major current instability was found, the possibility of a reoccurrence of landslipping of the Greensand and Gault over the underlying Fullers Earth had to be considered.

The critical factor controlling stability is ground water and this is affected by the balance between infiltration during rainfall, seepage from higher up the slope and transpiration by the trees. Geotechnical assessment indicated that the site was stable under conditions prevailing at the time of the site investigation but that water levels should be monitored over a period of time. Furthermore, buildings should not be constructed across the lines of slips – each building should be located entirely on one slipped plate. Extensive felling of trees should also be avoided and a transverse drain should be installed across the site to intercept ground water seepage.

Several trial pits were dug to establish the line of the proposed deep land drain. To fall across the site and follow the clay surface, it was found that it had to pass under the workshop. In this area the drain was backfilled with no fines concrete.

Standpipes were installed in the site investigation boreholes and water levels were monitored before and after construction of the workshop foundations. The deep land drain was found to be effective in minimising seasonal water level fluctuations.

The foundation for the workshop is a simple reinforced concrete raft slab. As far as possible the position of the workshop was arranged to fit in with the contours of the site but a small amount of cut and fill was necessary. On the downhill side, the raft was underpinned by a no-fines concrete wall to allow for future construction of a retaining wall above the visitor centre. Reinforced concrete internal walls were constructed down each side of the raft slab to support the timber arch members. On the uphill side, this wall retains the soil.

Foundations for the prototype house were formed by digging holes, placing rejected concrete manhole rings, positioning the A frame timbers and filling with concrete.

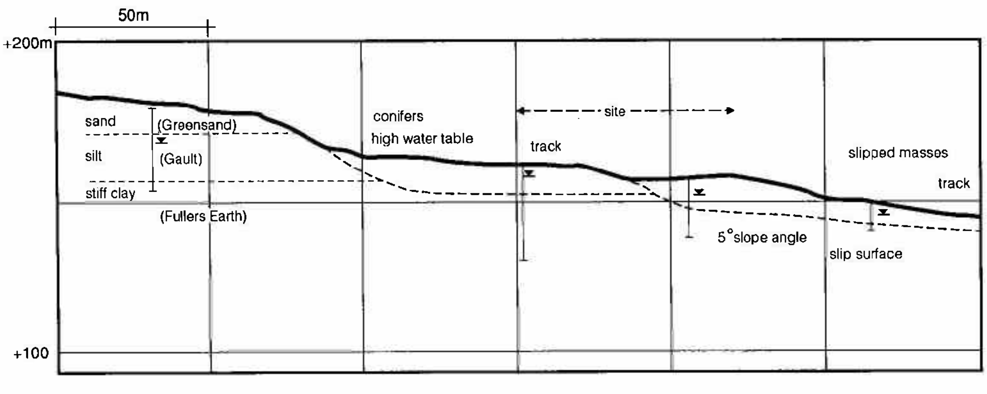


Fig 5.5 Geological section through site showing landslipped areas

The prototype house

The prototype house is 11.2m long by 8.5m wide and constructed of thinnings of roundwood. It has six rooms, each accessed by a central corridor (Fig 5.6). The roof structure uses thinnings 60–90mm in diameter at 450mm spacing to span a maximum distance of 5.5m. Due to their small diameter, the thinnings would be unable to support the expected loads in bending, but formed into a catenary shape with the ends restrained they act like cables resisting the applied load in tension.

Tension induced by applied loads is reduced with increased sag, and so it is structurally beneficial to maximise the initial sag. Stresses induced by the initial deflection relax to some extent with time and change in moisture content. When imposed loads (most importantly snow) are applied, additional stress induced will be superimposed on the relaxed initial stresses. Due to elongation of the timber and deflection of the supports an increased sag of some 50 to 100mm will be produced by maximum snow load.

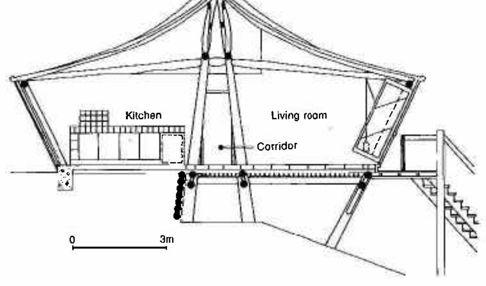


Fig 5.6 Section through prototype house

With the initial sag set at 200mm, the design sag for maximum load is 300mm which gives a tension force of 7.2kN. This tension is transferred into the building structure at the ridgeline through a tension joint attached to a wire cable hung between the heads of four A frames. The tension at the eaves is transferred into a cable spanning between inclined side posts.

The composite floor joists, spanning 3.6m at 450mm centres and made from two parallel timbers glued and bolted together, are supported on ladder beams formed by spacing parallel members apart with blocking pieces to create vierendeel type beams.

The roof is formed by an inner membrane of canvas which supports 75mm of Rockwool insulation covered by a randomly reinforced PVC coated polyester Sarna flat roofing membrane. This is coated with stone chippings which in due course will support the growth of moss and lichen.

Timber treatment

Spruce is a notoriously difficult wood to treat effectively by impregnation. In dip diffusion, timber is dipped in a supersaturated solution of boron salts and then stored under impervious sheets for four to eight weeks. The cells of green timber are full of water and boron can diffuse through the cell walls, penetrating to the heart of the wood. Because the method is only effective on green wood, it is rarely used in this country where the vast majority of timber is imported. This method of treatment was, however, undertaken on the Hooke project.

The main arch members were too large to dip in tanks and so were sprayed with the boron solution and then enclosed in polythene sleeves. The penetration of the salts can be tested destructively by cutting a section of timber and treating it with dye. Although the spraying technique was found to be effective, where timbers were cut the exposed surfaces required further treatment with CCA (copper chrome carbonate) applied by brush.

Tests were carried out to determine whether the salt could be washed out of dried treated timber. Results were encouraging but not extensive enough to prove that timber exposed to rain water could safely resist attack over a long period. All external poles were consequently treated by sap displacement using CCA.

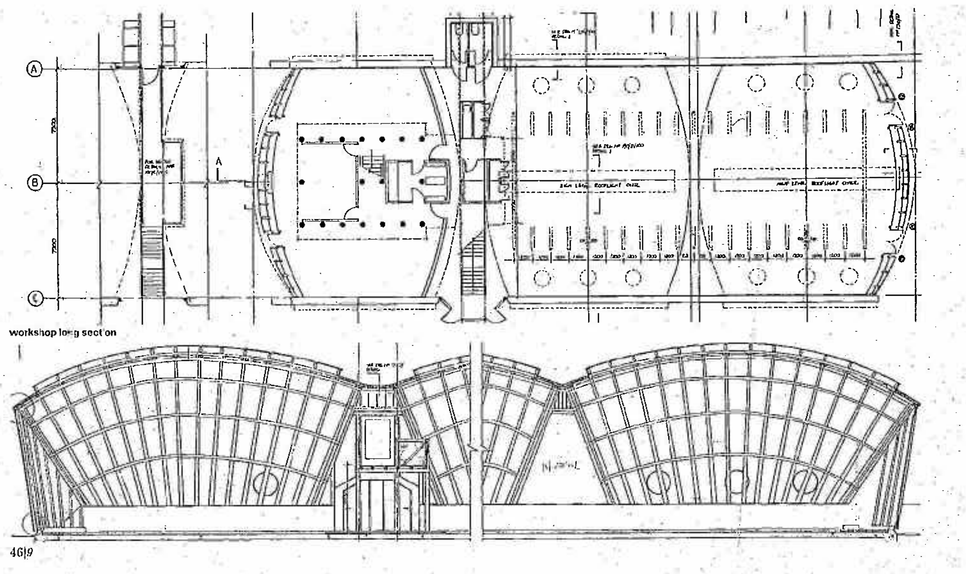


Fig 5.8 Plan and elevation of training centre

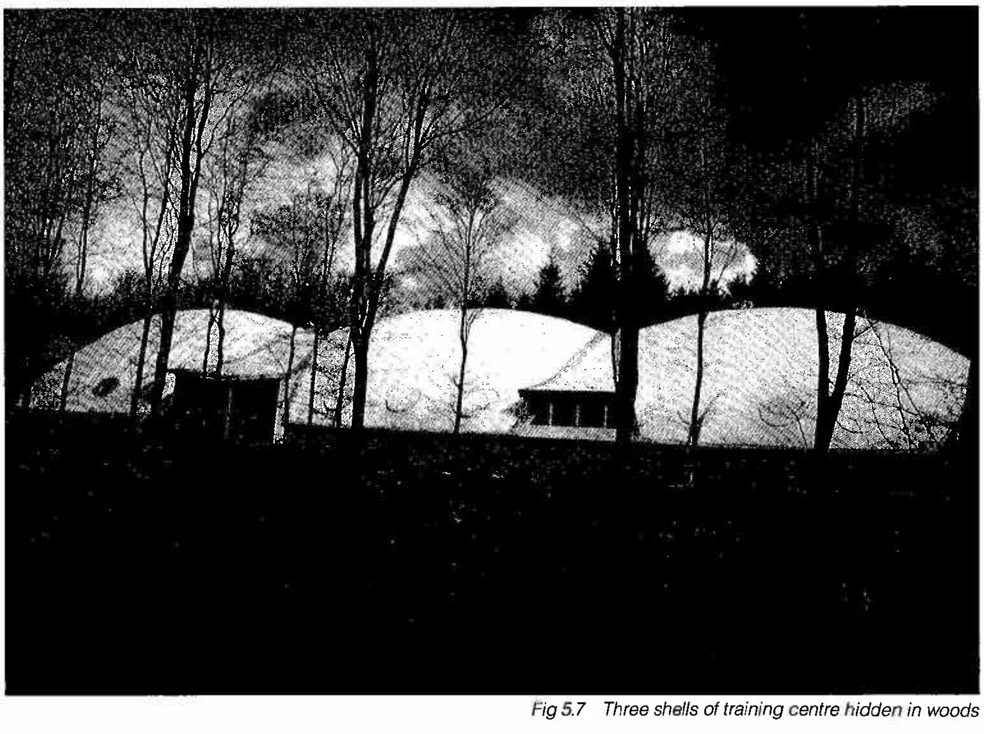


Fig 5.7 Three shells of training centre hidden in woods

Training centre

The proposed training centre was required to accommodate not only a large working space for machines and a separate teaching area at one end together with seminar rooms, but also offices and library and a viewing gallery for visitors – all constructed in roundwood poles.

The final building is comprised of three shells spanning 15m forming a structure 42.5m long and 7m high (Fig 5.7). The shells are formed using pairs of thinnings of nominal diameter 155mm at the base to 65mm, approximately 9m long joined at the crown by a laminated crown arch member. Two of the shells form the workshop area, the third contains the library, seminar rooms, teaching area and offices, all arranged at ground floor and mezzanine levels (Fig 5.8). Access for visitors is by a bridge which enters the building at mezzanine level between the workshop and teaching areas.

Development of building geometry

Computer analysis was used to predict the shape of a typical pole bent in cantilever when pulled down by rope. A trial arch was erected with a simple pinned crown and tests were carried out on numerous poles to check that the predicted shape and loads for cantilever poles approximated to those achieved with full size samples.

The pinned crown arch was easily formed and proved a good structural form as the pin reduced the bending moments at midspan where the section of the pole tapered to its smallest diameter. Its shape would however, vary considerably with the properties of the poles as the rotational direction of each pole can be different at the pin. Alternatively, using a moment connection at the crown, the arch is constrained to be horizontal at or near the centre of the building, and the shapes of the arches are consequently more consistent. For aesthetic reasons, a fixed crown was preferred.

The computer predicted shape could be achieved in each side of the arch however, at the crown, the radius of curvature required to smoothly join the two main arch members was too small for the end diameter of the pole, requiring the introduction of a crown arch piece. This element was fabricated by laminating two bent pieces together with glue and bolts. Before full production of these elements started, some test loading was carried out in the woods.

The geometry of the shells was formed by rotating the arches on the arc of a circle on the longitudinal centre line of the building. Longitudinal members are spaced evenly around the arches and the crossing point of each longitudinal member with each arch forms a node point for the geometry. The basic arch shape cannot easily be described mathematically, and the node co-ordinates were initially determined using traditional graphical methods. They were then entered into a computer for final smoothing of the shell and full nodal co-ordinates were produced.

The interstitial spaces between the shells are of unequal size. Between the workshop and the teaching area, the space is widened to accommodate the public gallery passing through the building. At the ends of the shells the arches are doubled up to provide extra strength to support the interstitial spaces and to enable the connection of window mullions at the building ends (Fig 5.9).

Arches spring from reinforced concrete upstand walls, which for architectural reasons, and to ease construction, are straight. Because of this the setting out of the fixing bolts sleeves varied from arch to arch – had the walls been circular in plan the springing points could have been standardised.

A simple model of the training centre (Fig 5.10) was made which clearly demonstrated that, despite the simplicity of setting out, a visually interesting shape derived from both geometric and mechanical

Fig 5.9 Window mullions and ventilation openings inside the training centre

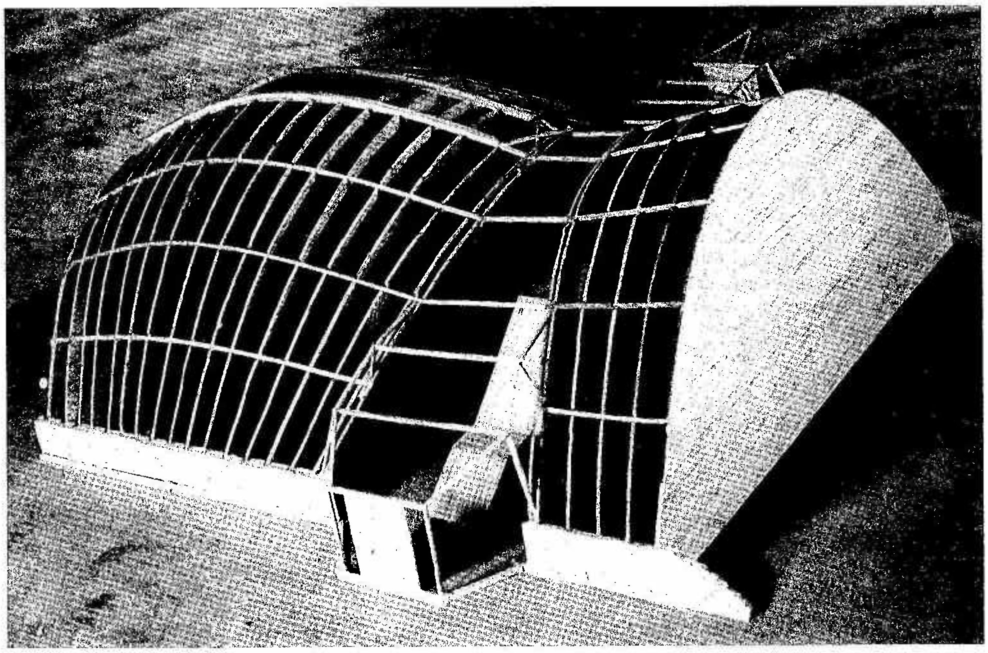


Fig 5.10 Model of training centre

properties of the timber poles would be achieved.

Bracing to the shells is provided by a PVC-coated polyester membane acting as the tension member of a cross braced system in conjunction with the arches and longitudinal members, particularly under asymmetric loading. The membrane is fixed to the main arches with 2.65mm stainless steel nails at 200mm centres and is laid on a bias to provide stiffness in the diagonal direction. A separate geometrical analysis was carried out providing full dimensional information of each element for the contractor, and patterning produced for the membrane.

The inner membrane of the roof is comprised of Sarnafil S521-05 Type 1, a PVC-coated polyester woven membrane which complies with DIN 4102 Class B1 for fire resistance. With this DIN rating, the membrane itself does not support combustion and, when impinged on by flame, burns and vapourises locally. This forms an opening allowing smoke to escape in the same way as at the Imagination Building (Ref 5.1). It was not known just how the insulation between the membranes would affect this process and so full scale fire tests were carried out under the supervision of the Fire Research Station. These confirmed that the performance under fire would be as anticipated.

A standard membrane edge detail with roped edge and aluminium clamp is screwed into two layers of 12mm thick plywood. Slabs of mineral wool insulation 75mm thick (Rockwool) fixed to the arches lie between this inner membrane and the outer membrane of Sarnafil S327 12EL, a randomly reinforced flat roofing material (Fig 5.11). Standard Sarna plastic coated metal pieces bent to shape and welded to the membrane on site are used as edge flashings.

At the rooflight the bracing, which in other areas is provided by the inner membrane, is continued using steel wire ropes. Either side of the rooflight, the ventilation opening is sealed by the inflation of a tube fabricated from inner membrane material.

Construction details and method

The construction of the building was intimately linked with its design. The construction method had to be thoroughly thought out at design stage and its implications accounted for both in terms of appropriate details and checking of stresses, before viability could be demonstrated.

Crown arch members

These were formed by cutting a round pole longitudinally, bending in a jig, and gluing and bolting back together to produce a laminated member with a radius of approximately 2.5m. When the glue was cured, a tie was put across the ends to hold the shape. The glue was considered as a filler, the bolts being designed to take all the horizontal shear load.

Longitudinal members

The longitudinal members on the workshop were intended to act not only as important, though lightly stressed elements of the shell, but also as spacers during construction, referred to as noggins. The joint between noggins and arches (Figs 5.12 a, b) although not a craftsman's delight in engineering terms meets a number of well defined requirements. The use of such a joint prevented fibres in the arch

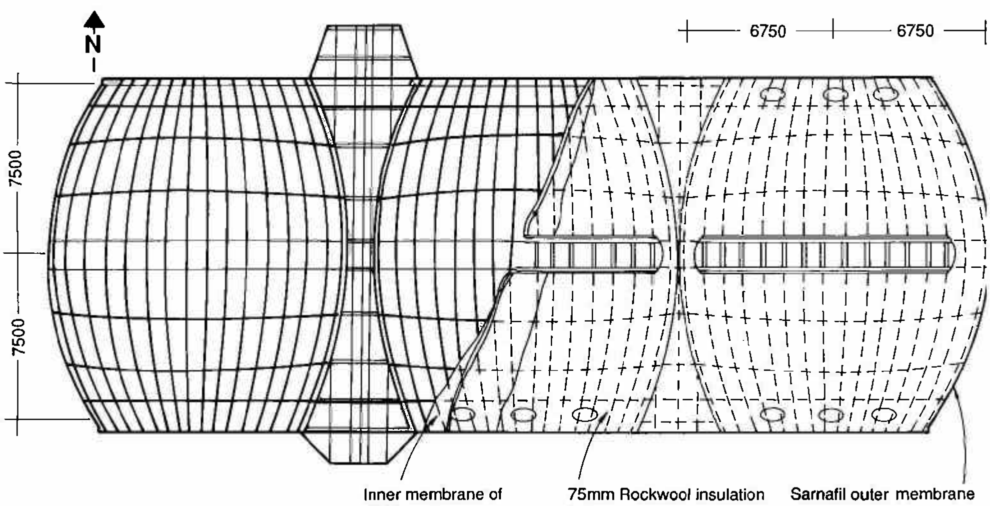


Fig 5.11 Plan of shells showing arches, windows and membranes of roo

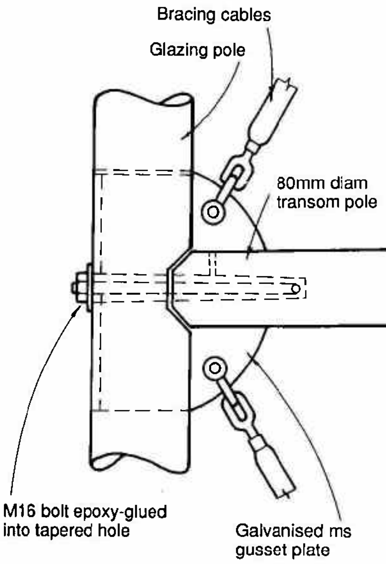


Fig 5.12 a) Section through arch noggin joint

To simplify the contractor's task in erecting the building, the main structural elements were conceived as a kit of parts slotting easily together. By bolting one noggin block to the noggin in its theoretically correct position during prefabrication, when the joint was made with the arch the other noggin block could be pushed tight against the arch and then drilled through the noggin. The steel strap over the top is thin enough to be bent to shape to suit. The joint formed by drawing the strap and arch down against the slightly tapered blocks is a tight fit and has some fixity which provides redundancy to the structure and increases the stiffness and the structural factor of safety.

Arch members

The arch members are the most highly stressed members and, as the primary elements for carrying imposed loads, particular care was required in cutting these timbers. They are the largest members in cross section, taking several years to dry out and metal should not be embedded in them. Consideration was given to prebending the members and stockpiling, but this was not

As soon as possible after sufficient analysis had been carried out to size the members, a second trial arch was erected to prove the viability of the fixed crown. At first it was found impossible to bend the timbers to the required shape – they broke before being bent over far enough at the top to splice in the crown arch. It was decided that to provide extra flexibility at the lower end of the arch and to regularise the size of the timbers at their base, a slice would be taken off the arches on their lower (compression) face. This went against the principle of not cutting the fibres but provided reduction in stiffness at the springing points giving an even distribution for the maximum stress envelope. This proved most successful and a trial arch was completed using a carefully thought out procedure for installing the crown arch.

The springing point detail makes use of the flat face produced by removal of the slice (Fig 5.13 a, b, c),

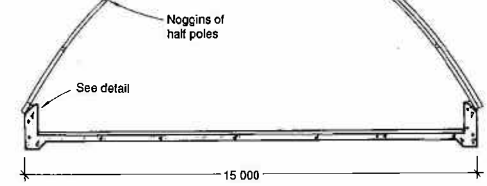
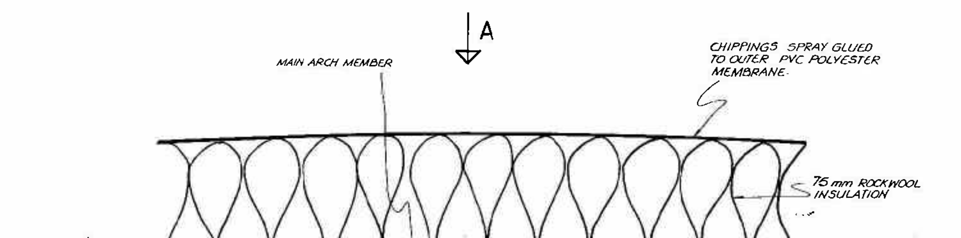


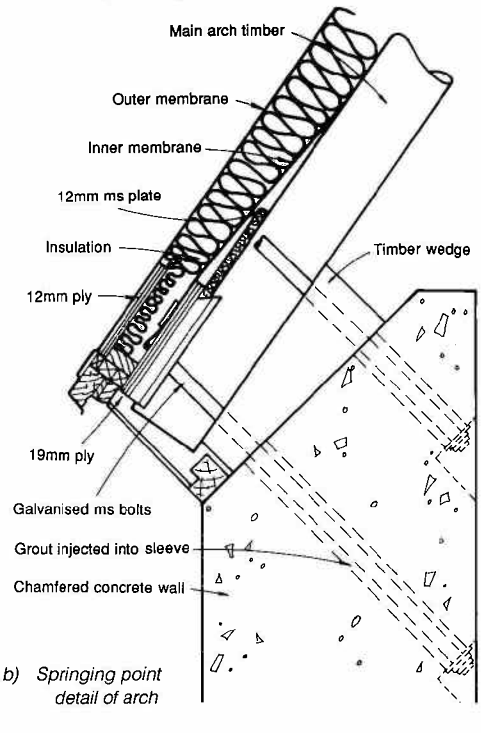
Fig 5.13 a) Section through arch construction



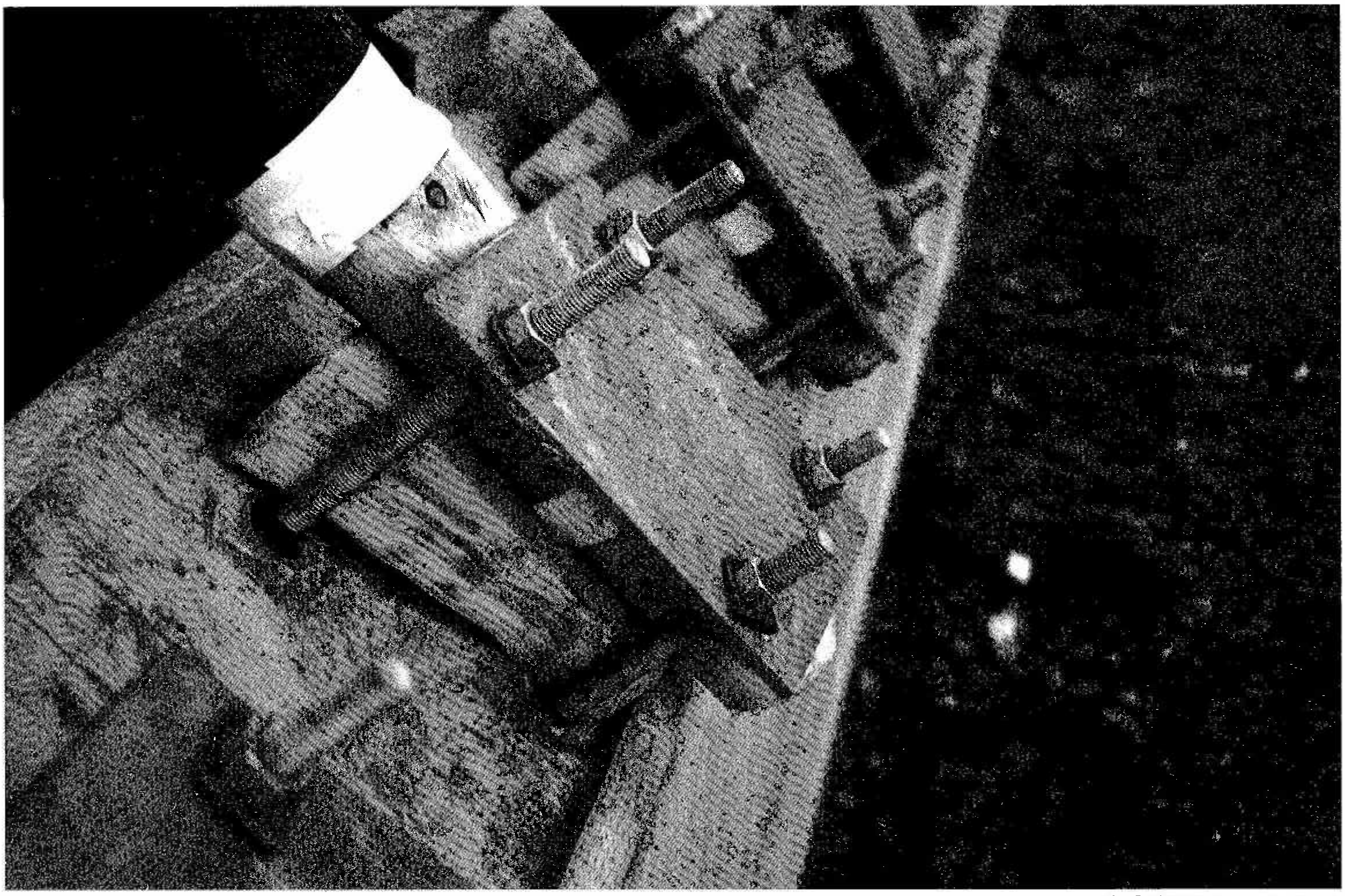
b) Typical end wall bracing detail

member from being cut, and noggins could be accurately marked before assembly to show the node positions. As each arch member varies in size, this type of joint would allow for considerable variation in arch size, and much of the joint could be prefabricated to avoid working in the air.

desirable or feasible. The timbers should be as green as possible to minimise forces required to bend them over. The springing point joint, the first joint to be made, needed to be both accurately set out and yet quick and simple as the timber would need to be held in the air whilst the joint was constructed.



b) Springing point detail of arch



c) Springing point detail of arch

The reinforced concrete upstand wall has its top face cast at 45°. The angle of each arch was calculated and a wedge detailed to set each arch at its required angle to the vertical. The arch timber is held in place by a flat cover plate with four bolts which pass through oversize sleeves cast in the wall. Oversize holes in the wall were set out to suit the horizontal angle of the arches and so when bolted in position with its correct wedge, each arch would be set in its correct position with some tolerance for adjustment.

The completion of the trial arch proved the viability of the system and full scale erection could proceed.

Construction period

During construction there was some concern over the possibility of debarking the main arch timbers throughout the period of the project. It was known that they could be debarked by hand in the spring and early summer but it was thought that when the sap stopped rising it would be necessary to use mechanical debarking, damaging the important outer fibres of the timber.

For this reason a large stockpile of main arch members was prepared and by the time full scale erection could proceed some of these were three months old. Many of these timbers were broken when stressed. In construction of the first shell there was 60% wastage, requiring replacement with fresh timbers. Fortunately it was found that debarking by hand was effective even into December, and a technique of overbending timbers and then releasing slightly was discovered to be more effective than the earlier more timid method of gradual slow bending. Replacement of timbers during bending was then reduced to only 10%.

All the main elements were fully defined on the drawings and node points were marked on them before erection. The noggin blocks set the arch in the longitudinal direction and although site setting out was more difficult than for an average project, it was found that once the arch members had been carefully set up at the springing points, only one co-ordinate needed to be monitored as the node was set into position. Post-erection survey checks showed that node positions were within a tolerance of 25mm. On the lower portion of the arch it was necessary to use props to push the arch out to achieve the required shape.

Ties and props were left in position until all the crown members had been spliced in and the glue had cured (Fig 5.14). Little movement was experienced as the complete shells were untied, starting in the middle and working outwards symmetrically.

Despite early setbacks, work with the roundwood proceeded very quickly once the optimum technique had been established. On the third shell, when a breakage occurred towards the end of the bending process, it was possible to fell, debark, boron spray, mark node points, cut the slice and sleeve a tree in one day – erecting it, bending it into position and fitting the crown splice on the next.

Fig 5.14 Construction of crown arch members

The bending process was an unusual sight (Fig 5.15) but once the techniques had been learnt the framework was successfully completed (Fig 5.16).

The most labour intensive part of the work involved fitting the plywood edging strips for the membrane, and fitting the windows and doors to the complex shapes of the frame. This work required several skilled carpenters and took several weeks.

Building services and environmental engineering

The college occupies an isolated site, 1km distant from mains power and with mains drainage connections impractical. Building services solutions thus needed to address these problems in addition to providing an energy efficient design.

The heating concept required forest products to be used as fuel to minimise the outside dependence on energy. A wood burning back boiler stove is used in the staff accommodation, while an energy centre containing two wood-fired boilers was to provide the heating requirements of the workshop and future phases. The foundations for the energy centre and its flue tower were constructed but the capital cost of construction of this facility and purchase of the boilers exceeded that of a proprietary self-contained oil fired boiler package. The latter was reluctantly chosen with a view to building the energy centre at a later date.

The heating of the workshop is by a Wirsbo underfloor heating system which uses low temperature hot water circulated in a system of plastic pipes cast into a concrete screed. Heavy duty insulation is provided between the concrete screed and the raft slab in the workshop area. A pattern of plinths to support workshop machinery was created by omitting regular strips of insulation and placing the concrete topping in contact with the raft slab.

Ventilation is provided by a vent either side of the roof light at high level and by horizontal openings under the eaves of the shell roof at the springing point position. The low level vent can be opened and closed by a simple flap door secured by a rope.

A new electrical supply was taken from Burcombe village, 1km distant from the site. Following difficulties in obtaining wayleaves for a cable route, compulsory wayleaves were eventually obtained for sections of the overhead and underground route. Elements could only be hand dug as access for conventional mechanical machinery was impossible. At a point 40m from the workshop, the cable connects to a 200KVA pole mounted transformer which feeds via underground ducts to the mains distribution board within the workshop building. Facilities are also included for distribution to later phases of the development. Electrical distribution by a three phase suspended plug-

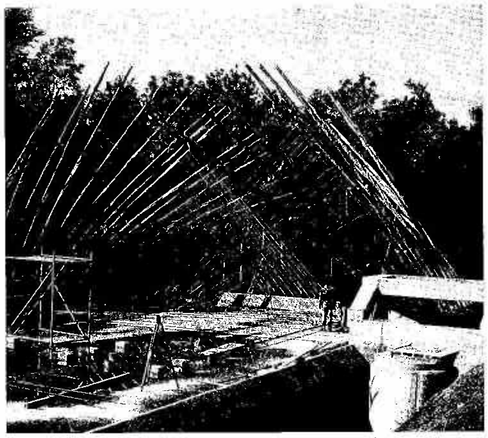


Fig 5.15 Bending of timbers during shell construction

in bus bar serves the larger machinery in the workshop centre, while a 13A supply is routed around the perimeter in trunking to serve the benches.

Lighting to the workshop is by a combination of natural daylight, calculated to achieve a minimum daylight factor of 2.5% through the rooflights and glazed ends, and fourteen high bay luminaires using 250W SON de-luxe lamps selected for their particular rendering properties when working with wood.

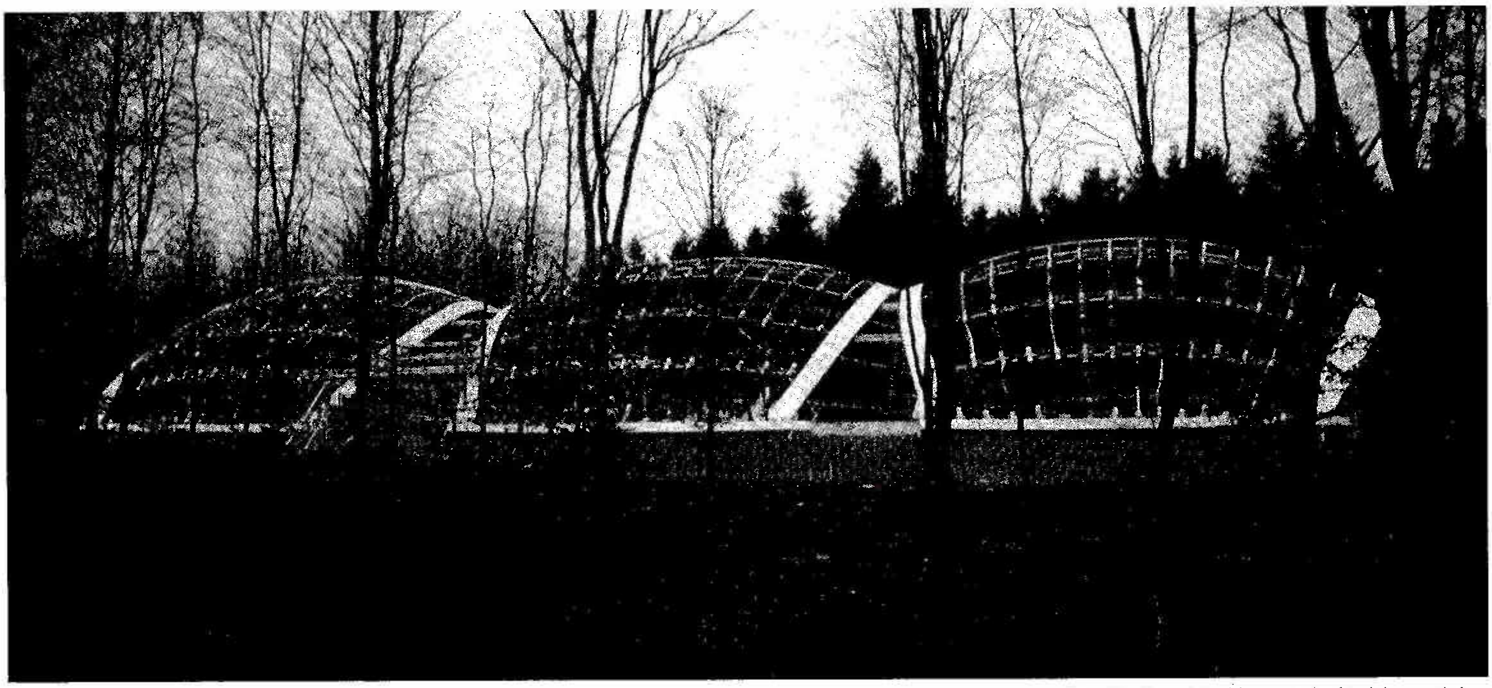


Fig 5.16 Completed framework of training workshop

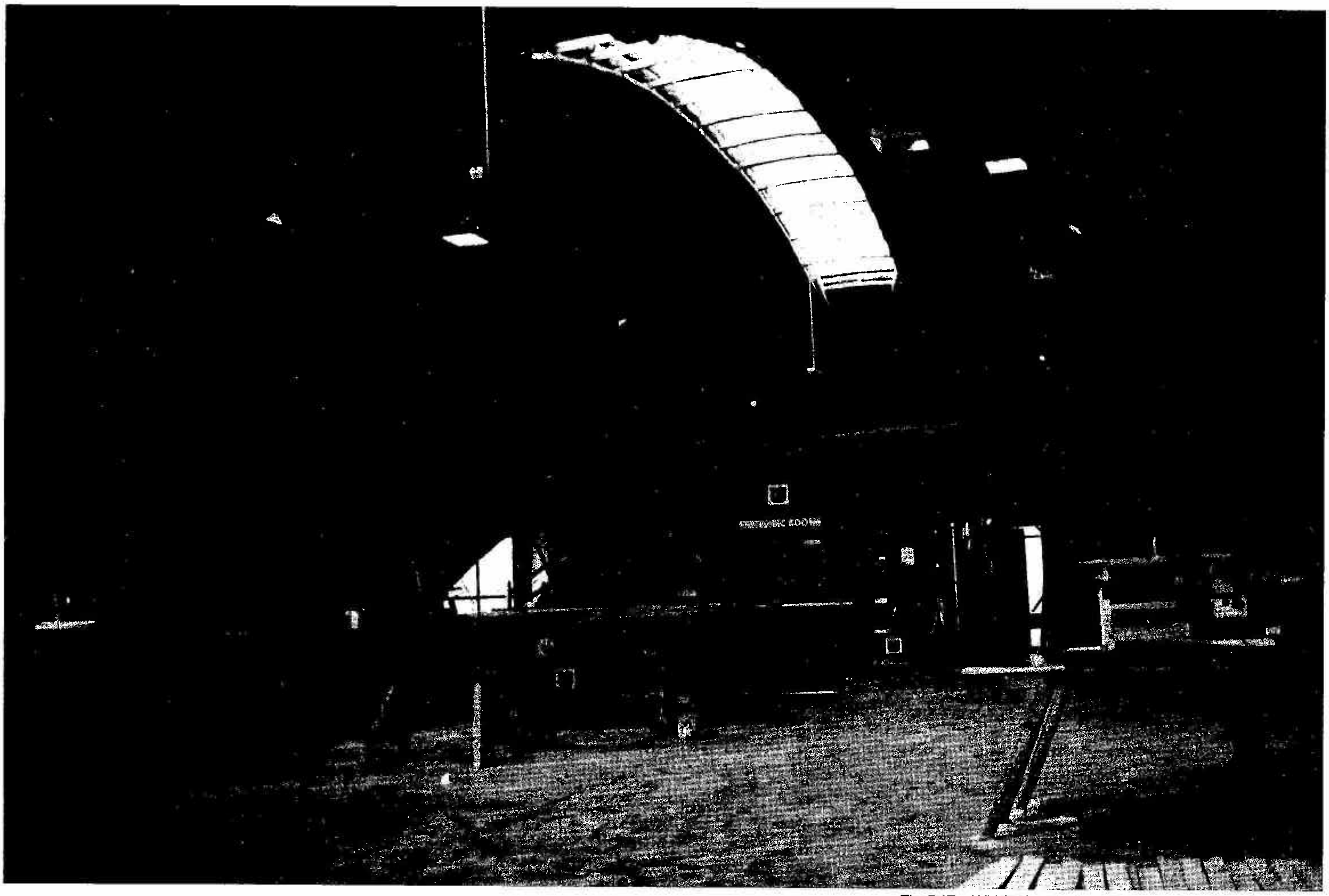


Fig 5.17 Within the shells of the completed training centre

A small stream flows between the workshop and house and storm water is carried in land drains to this stream. Foul drainage is taken to a Klargester Biodisc, a small, electrically driven packaged sewage treatment plant contained in a glassfibre tank. Sewage is treated here and discharges into a large herringbone soakaway near the stream.

The college in use

It can be reported that in the storms of January 1990, despite winds strong enough to extensively damage much of the surrounding woodland, the buildings performed well. The prototype house is now being used as offices and seminar rooms for the college and the workshop building is fitted out with woodworking machinery (Fig 5.17). The buildings will need to be properly cared for, but if maintained, there is no reason why they should not be durable and perform well for many years. It is not possible to predict every quirk of a natural material's performance but we believe that the buildings prove that the use of roundwood thinnings in construction is viable. On the basis of energy use per unit stiffness, timber is economical in its usual sawn mature wood form (Ref 5.2). Immature thinnings used close to their source are very attractive in terms of their impact on the environment. Sufficient information has been gathered in the test, design and construction processes to enable buildings to be constructed in the future with a better appreciation and knowledge of their performance.

A most encouraging development is that graduates from the college have now built houses to commissions from other organisations who also recognise that for forests to be conserved, they must be made to work. It is this expansion of the principles founded at Hooke Park College that will encourage the use of roundwood thinnings to thrive.

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References

5.1 Cook M, Liddell I, and McLaughlin T. "New headquarters for Imagination, Staffordshire House, London". Patterns 7 pp 5–10, Dec 1990.

5.2 Biggs B. "A full repairing lease" Patterns 7 pp 31–32, Dec 1990.

5.3 Happold E, Dickson M and Sutherland I. "A school for Woodland industry" Procs. of Int. Conf. on Design and Construction of Non Conventional Structures. ISE and IASSS, 1987.

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