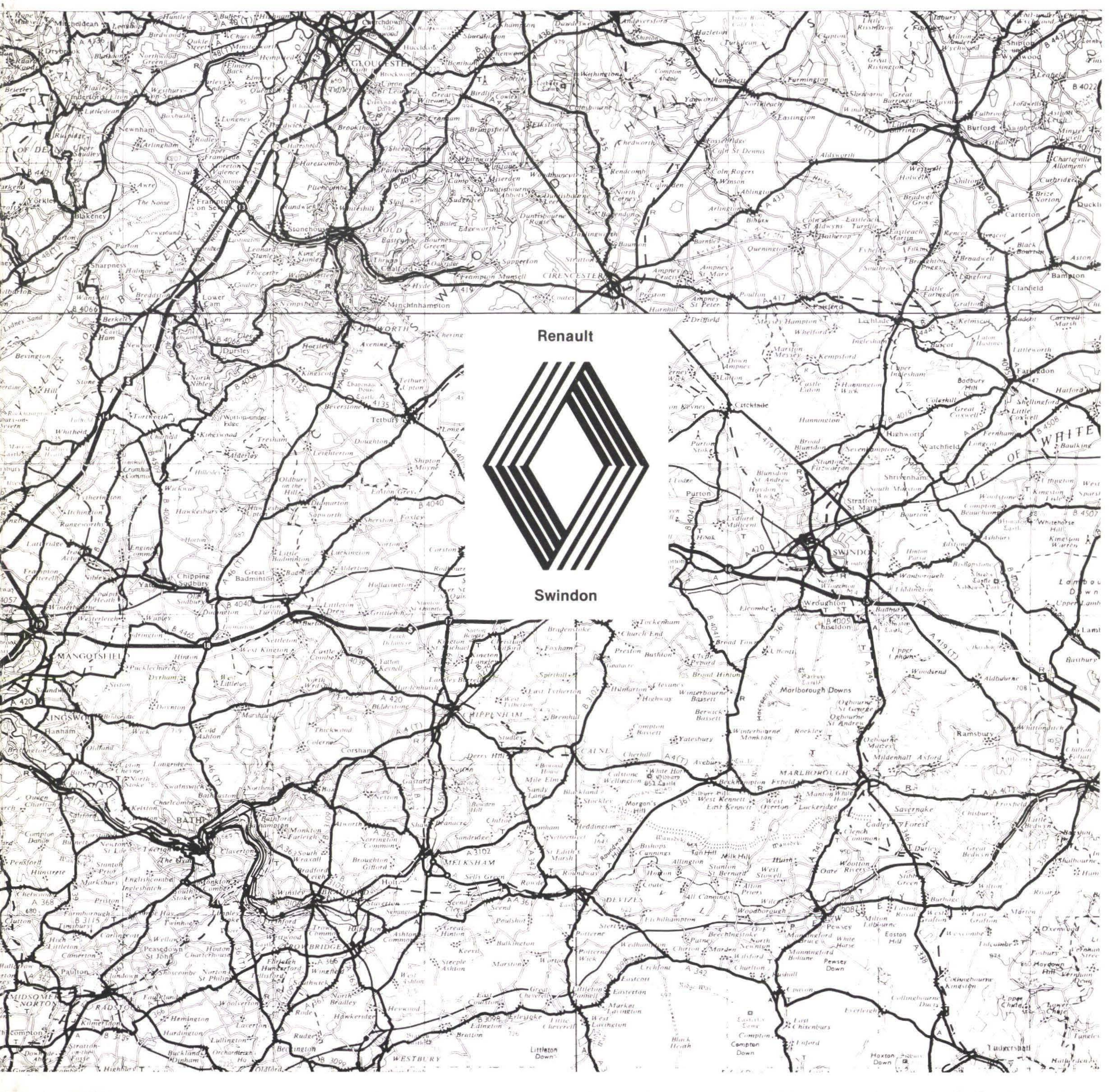
THE ARUP JOURNAL

SEPTEMBER 1983



THE ARUP JOURNAL

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Architect: Foster Associates

Martin Manning

Introduction

Renault, Swindon, can perhaps be seen as a member of a family of steel roof structures on which the firm has worked over the past few years. Its form and details derive from considerations of structural behaviour to minimize the size of the main components. Some of the other members of this family are the Cummins Factory at Shotts, the NEC Hall 7 and Fleetguard Factory at Quimper. John Thornton has written about the family's characteristics elsewhere.

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

When we were invited to work on the project by Norman Foster, the client had already obtained planning permission on the basis of a design and construct package deal for a fairly conventional warehouse. The proportion of the site which was to be developed was 50%. At about the same time the Directorate of Renault in Paris were considering, as Cummins had in the States, the advantages which might accrue if their buildings were designed by good architects. After a series of interviews in this country Norman Foster was appointed to prepare a feasibility study for the scheme.

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

The professional team was appointed to prepare the design in February of 1981.

The general arrangement

The brief required 20,000m^2 of warehouse, a training school, a showroom and an office, totalling in all some 25,000m^2. A principally single storey building with a footprint of that size would clearly imply a substantial earthworks contract on any site.

The site was a green field to the west of Swindon with a fall across it of about 5m. We had been able to carry out a site investigation in late 1980 to establish the sub-soil conditions which were top soil over alluvium over Kimmeridge Clay.

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

Considerations were also made of the appropriate module size for a multi-use building of this kind. The sizes of the stored pieces in the car industry do not lend themselves to automated storage systems.

Such warehouses operate on the principle of a staffed supermarket and require the facility to move racking and storage about throughout the life of the building. With the requirements of the showroom and training school it became apparent that a module size of the order of 24m square with an 8m internal height would be appropriate for such a development.

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

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

The steelwork

The form

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

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

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

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

Uniform downward loads stress the tension members. Sway forces or wind uplift forces destress some of the tension members. The resistance of the structure to horizontal disturbance is then small. The first aim was to define the members permanently available. We chose to make the structure sufficiently stiff so that modes of failure due to instability would not interfere with yield and proposed that from Merchant-Rankine, ratios of (Pcrit/P) in excess of 10 would be sufficient. To achieve this the ties adjacent to the masts were prestressed to provide not only a moment connection between the beams and columns but also to strengthen and stiffen the columns themselves. The main span tension hangers are principally activated by downward load.

The analysis

As each loadcase selects which of the non-prestressed tension members are destressed and therefore which are eliminated from the structure, analyses were carried out for each of the worst arrangements of load. The worst loading was associated with a 1:50 year wind, a dominant opening on the windward face and an asymmetric distribution of live load. Deflections were kept to those suggested in *BS 449*, both for sway and beam movement. The effects of a 20°C temperature change were included. The section justification was carried out in accordance with *BS 449*. The factor of safety against destress in the prestressed members was chosen to be in excess of 1.5.

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

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

(2) Do they gallop downwind?

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

Detail design and fabrication

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

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

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

Spheroidal graphitic cast iron acts in a similar way to Grade 43 steel but because it is a material specifically designed to be cast, unlike cast steel, it is much cheaper not only to cast but also to heat-treat and to machine.

For quality assurance of the castings we chose to manufacture prototypes and perform a variety of tests. Firstly, the architect could confirm that the shape he had drawn and modelled was actually what he wanted and that the surface finish was acceptable. Secondly, we were able to assess, on pieces made by the particular founder's method of casting and control, the likely defect size, the degree of nodularity and ferriticity, the crack opening displacement (COD) value of the material, the mechanical properties (in terms of yield stress, elongation and Charpy values), the ladle analysis and the velocity of sound which is a measure of nodularity. By load testing the strength of the piece could be confirmed. There is some discussion as to whether COD tests in cast iron are really appropriate. It appears that brittle fracture in spheroidal graphitic (SG) irons just does not occur, regardless of the thickness of the piece or the temperature to which it is taken. However it provided a way to relate acceptable defect size to level of stress to material quality at a time when the firm's knowledge of the material was comparatively undeveloped.

The quality control method used during production was to compare the results of the mechanical tests on test bars, the ladle analysis and to check each piece ultrasonically for defects and velocity of sound.

The castings were connected to the columns by single pins through vertical plates welded to the wall of the column. In the

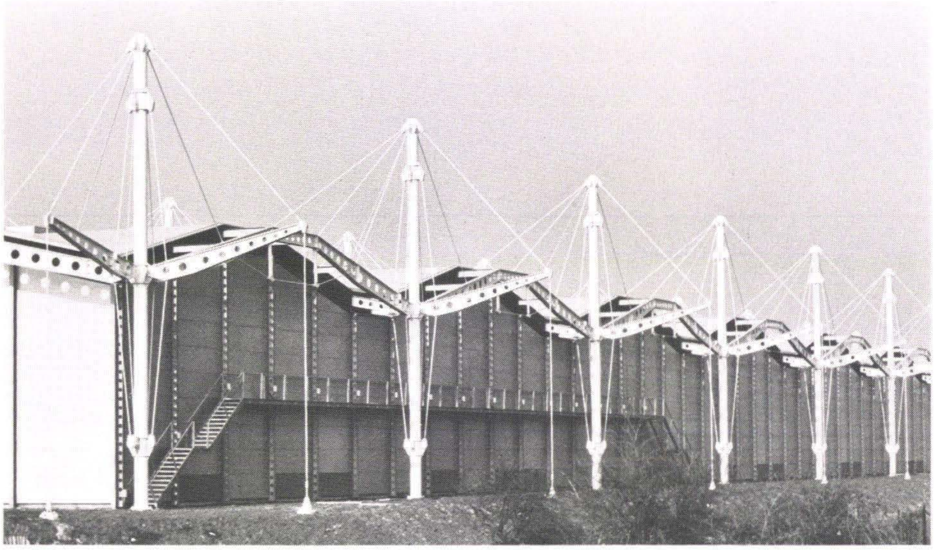
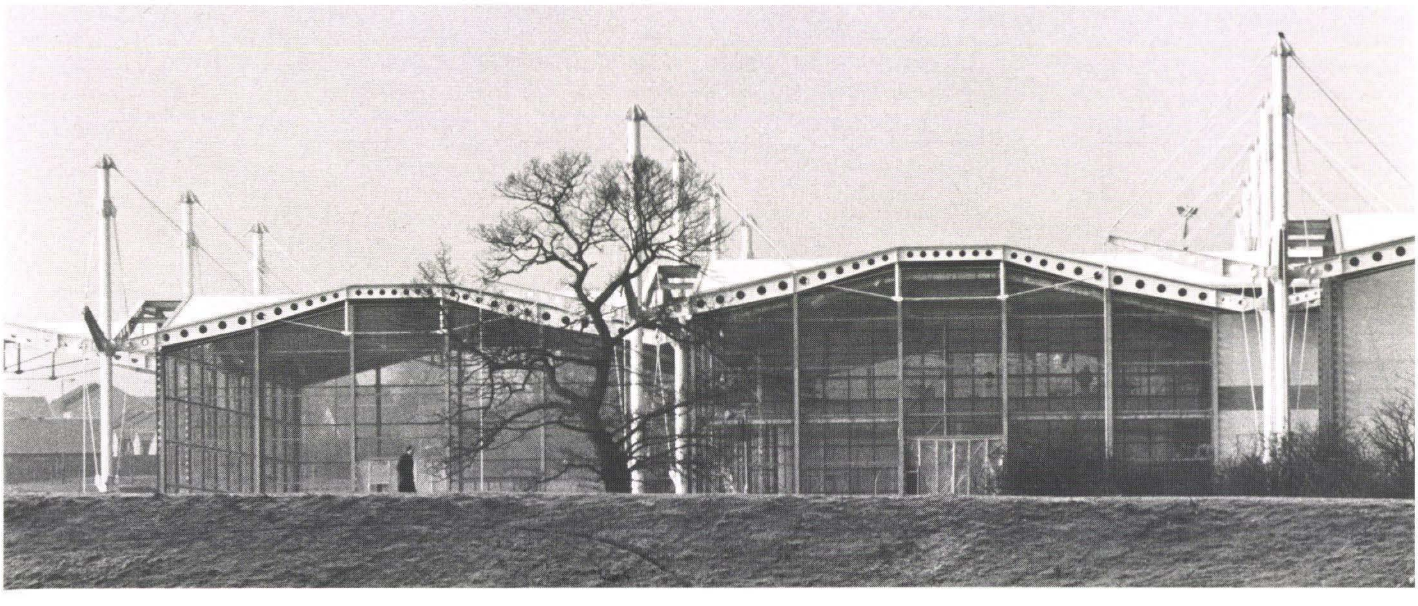


Fig. 1
The fire escape walkway and stairs which cantilever from the wall mullions allow the use of multilevel racking in the warehouse.

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



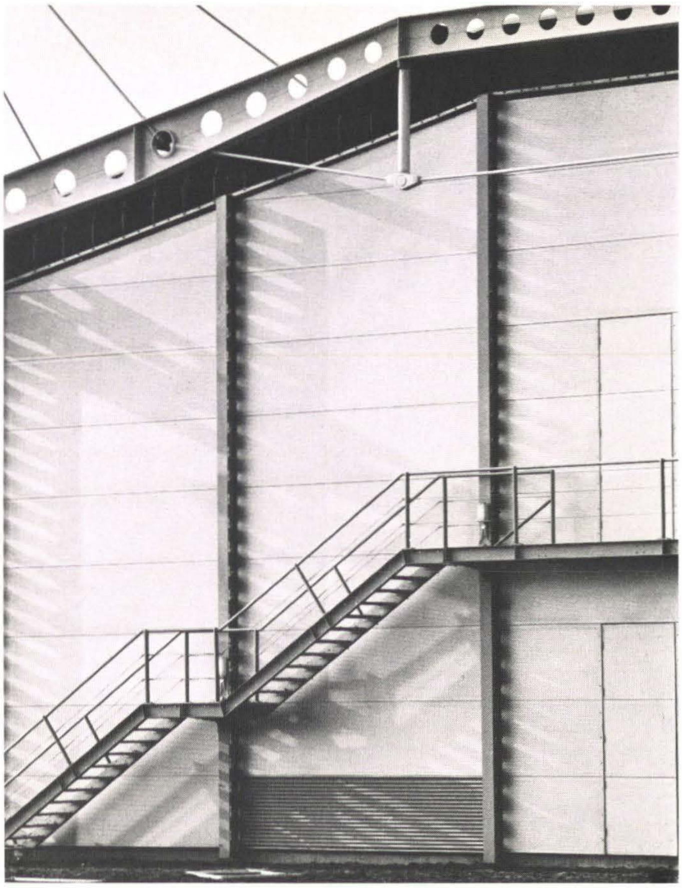


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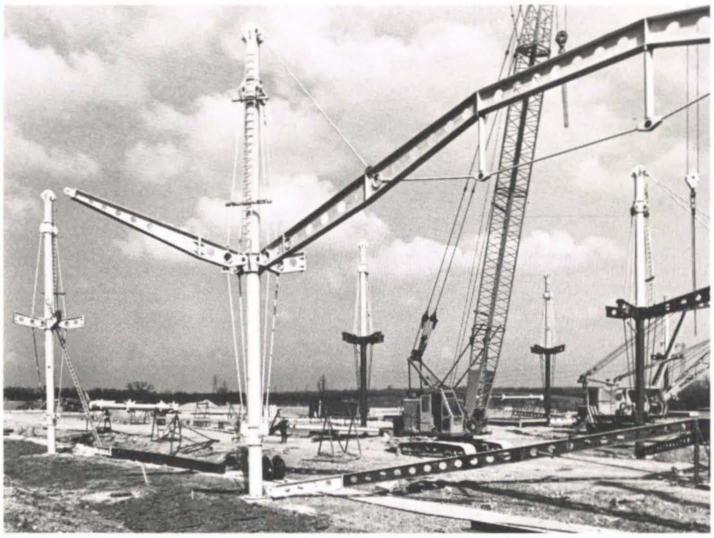
Fig. 3
The panels span horizontally between the mullions; the mullions span vertically between the slab and the roof; the roof moves vertically and horizontally in the plane of, and relative to, the wall.
The joint is made with a tensioned neoprene skirt.

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

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



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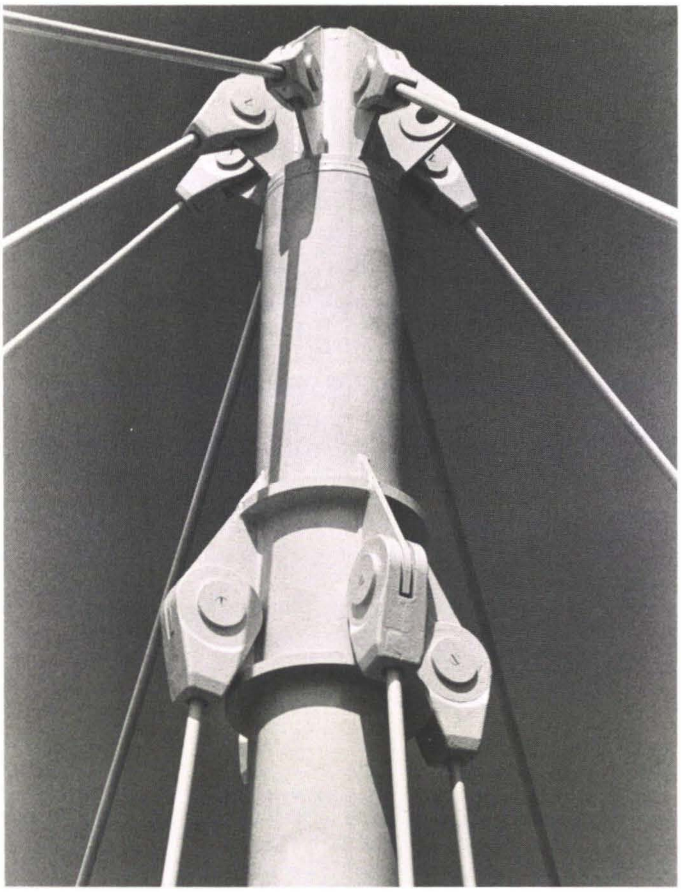

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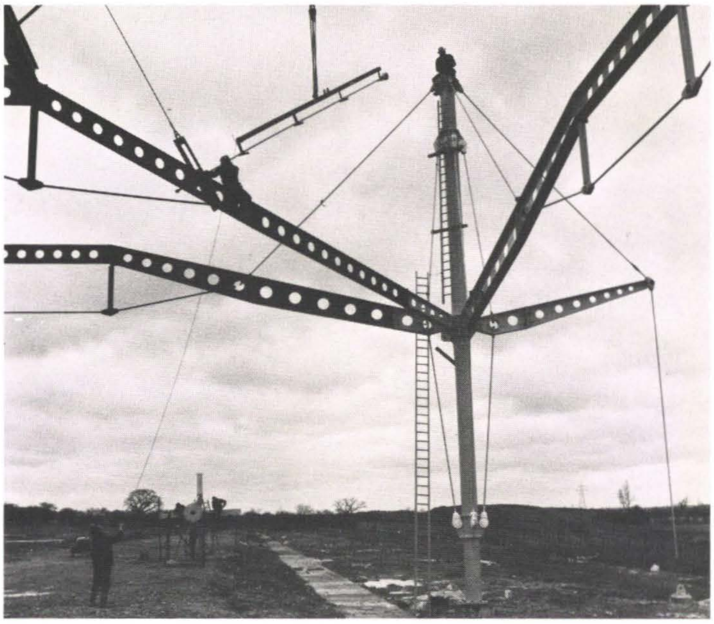


Fig. 6
Foam-filled silicon-sealed, spheroidal graphitic cast iron fork connectors with a working load of 60 tonnes for just under £100 each.

tender design these plates passed through the column. All stiffening to the section was provided internally. Because of the requirements of the programme and the difficulty of obtaining tube, the management contractor had decided that the columns had to be ordered before the steelwork sub-contractor was appointed if the project was to proceed to programme. As a result of the tender it became apparent that the cost of the details within the column was high and that all stiffening to the column should be external rather than internal. The design was changed so that the vertical plates were restrained by annular rings placed around the column. Such a configuration of course is exactly that most likely to induce lamellar tearing in the wall of the column. To enable us to assess the sub-contractor's welding procedures, we tested samples taken from the columns to determine the through thickness properties of the material and compared the results with published information to establish the likelihood of lamellar tearing taking place. Even though many of the lengths of tube had been made from the same billet, the values of the short transverse reduction of area (STRA) tests were all different. Thus, the distribution of non-metallic inclusions within the walls of the column was almost impossible to predict and lengths of tube could not therefore be selected to eliminate the defect. However the sub-contractor clearly had a problem in fitting annular rings onto tubular columns. The 457mm diameter columns varied by as much as 32mm across orthogonal diameters. A full strength butt weld between the wall of the column and a 40mm thick annular ring would produce a huge weld where the gaps were greatest. To overcome the problems of both tearing and tolerance the sub-contractor suggested buttering the surface of the column with weld metal before fitting the annular ring to ensure a precise fit. All of the welds in these columns were ultrasonically checked and no lamellar tearing took place.

The erection
By early 1982 the first pieces of steel had arrived on site.

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

The main grid line beams and the diagonal beams were assembled on the ground with their lower tension trusses only. They were lifted into position between the columns and the single pins inserted. The ties from the top of the column to the quarter span points on the beams were then attached to a strong back, set to the exact straight line length and placed in position. The nuts on the ties within the beam were done up finger tight.

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

On Renault the angle of inclination of the tie, the weight of the tie, and the tension induced in it by the weight of the steelwork alone are such that the ties are sufficiently straightened so that their load extension characteristics are very nearly linear elastic.

The weight of steel in the roof is similar to many other projects of this nature. Approximately 25,000m^2 of roof use 1400 tonnes of steel with some 27% being in the connections. Steel with such heavy details does incur some cost penalty with an average cost of £1150/tonne at September 1981 prices.

The earthworks and concrete

The earthworks
The boundary between the areas of cut and fill bisects the building along the diagonal.

Following the removal of the topsoil and the upper part of the alluvium a granular drainage blanket was laid over the entire area to be filled.

Excavated material from the area of cut was recompacted in the fill area, the criterion for suitability being moisture content.

On completion, land drains at 12m centres were installed in the area of cut and the whole area laid to falls to facilitate surface water runoff.

The earthworks protection
The roof covers a concrete floor with a high tolerance mono grano finish which is hard, clean and non-dusting. Such a slab has to be built under cover if curing rates are to be controlled successfully. Such a slab clearly cannot be used for access for the construction of the roof.

The sequence of construction was earthworks, foundations, roof and roof cladding, and then the concrete slab. However, while the foundations and roof are being constructed a method had to be found of protecting the excavated, and recompacted, clay over the winter while construction traffic ran on the top of it. Three ways were considered.

(1) The excavated area can be left high and the area of fill overfilled with the surface laid to falls to a drainage system. Care must be taken with construction traffic.

(2) The clay can be cut and filled to some final level and the whole topped off with a thick layer of granular material which would form not only the construction surface but also the final hard core under the ground slab.

(3) The site can be covered, after the earthworks contract has been completed, with a layer of lean mix concrete sprayed with tar to provide a waterproof surface which successfully protects the clay beneath from construction traffic.

Following comparisons of performance and cost it was decided to use the method of clay overburden and to lay construction roads as granular material on terram.

The contractual issues associated with the roads, which were laid by the foundation sub-contractor on subgrade for which he was responsible contractually in terms of performance and protection, these being used by a steelwork sub-contractor, all within a Management Contract, was clearly a risk. The condition of the sub-grade was monitored weekly at a grid of points across the site using a hand shear vane and relating the results to probable California bearing ratio (CBR) values.

Once the earthworks and foundations had been completed and the steelwork and the roof cladding erected, the roads and clay overburden were removed, the clay subgrade tested, granular material compacted, and the slab cast.

The protection system worked well and damage to the clay subgrade was found rarely.

The concrete
The purpose of the slab is to support a steel racking system with precise tolerance requirements. We specified those usual with a high tolerance floor. The sub-contractor claimed that the tolerances specified were not possible with the width of bay we had also specified. The C & CA tested a sample panel and showed that the sub-contractor had in fact achieved the specification. The C & CA were rather sceptical of this because they have just produced a new guidance note on reinforced concrete floors where the best possible tolerance is defined and is well outside that conventionally specified for high tolerance floors. The construction of the slabs went well and has performed adequately.

In the southern part of the project there is an area of suspended concrete slab on which the Regional Office is located. The construction is a trough floor formed with purpose-made GRP moulds, the concrete being left exposed.

This concrete structure, as well as the roof structure, are founded on reinforced concrete pads founded on the virgin clay.

Other work

As well as the structure of the building we were responsible for other elements of the project such as the roads, car parks and surface water drainage systems and the mechanical and electrical engineering within the building. The level of servicing is comparatively low. At the time of writing, the work is still being completed on site. It will be described elsewhere.

We also assisted Foster Associates in the development of the cladding design which they prepared.

The wall cladding was developed specifically for the project. It is composed of 4m long double skin metal panels which are foam filled. The panels span horizontally between vertical mullions, at 4m centres to which they are attached with self-tapping screws, the gap being sealed with a neoprene strip. The mullions span between the ground floor slab and the roof but are only restrained at the roof normal to the plane of the wall. The roof is free vertically and horizontally in the plane of the wall. The gap between the roof and the top of the wall which has to accept these movements has been sealed with a pre-stressed nylon-reinforced neoprene usually used in hovercraft skirts.

The roof decking is a metal deck connected to the purlins in such a way that their compression flanges are restrained against lateral buckling. Insulation is laid onto this and the overlying waterproofing system is a single bonded Trocal sheet. The total assembly was specified as having a self-weight not less than a minimum value to reduce the effects of uplift due to wind.

Rainwater drainage from the roof is from the base of the inverted frustra down a pipe within the column. The geometry of the roof is used to advantage in that under extreme rainfalls water head is allowed to build up in the frustra so that flow through the outlets transfers from weir to orifice flow and the volume which can be taken down the pipes is increased.

The contract method

The professional team was appointed in February 1981 and handover of the building was programmed for December 1982. The cost of the project to September 1981 prices was approximately £8½m. With such a programme and the team's desire to enlist the sub-contractors' expertise in the development of the detail element design, it was necessary to integrate the design and construction phases. This implied the use of a management contract. Bovis Construction were appointed in July 1981 to act as the management contractor.

By that stage the professional team had defined the principal packages for the civil and structural work and had already gone out to tender for the earthworks package. The successful tenderer was Isis Construction Ltd.

The steelwork tender was sought in August 1981 on drawings showing the layout and sizes of members, the typical details and a specification of materials, workmanship, performance and tests. The contractor was asked to develop all the details on the structure from our typical details, prepare fabrication drawings and welding procedures, and to submit them to us for our approval. No Bill of Quantities was issued with the tender documents but the tenderers were asked to measure and price a bill to a set of rules established by the quantity surveyor. This provided sufficient rates for the quantity surveyor to carry out any negotiations on variations in the contract. The successful tenderer was Tubeworkers.

The foundations, sub-slab drainage and high tolerance ground slab were let as one package and the reinforced concrete superstructure as another. In the event the same contractor, Cementation Construction Ltd., won both.

The warehouse section of the project was handed over to the client on programme in December 1982, the offices following during spring 1983. The final project cost will be within the budget set in autumn 1980.

The building was officially opened by Madame Lalumiere, the Minister of Consumer Affairs in the French Government, on 15 June 1983.

Credits

Client:
Renault UK Ltd.
Architect:
Foster Associates
Quantity surveyor:
Davis Belfield & Everest
Photos:
Harry Sowden

St. David's Hall, Cardiff

Architects: J Seymour Harris Partnership

Ian Fenner

History

St. David's Hall, the new National Concert and Conference Hall for Wales, is situated in the heart of the commercial centre of Cardiff. The site on which St. David's Hall now stands was originally allocated to the new City Library under the original master redevelopment plan for the City Centre. Ravenseft Properties, in partnership with the city of Cardiff, put forward, in the early 1970s, proposals for the redevelopment of what was by then a decaying city centre; an area of the city which was in stark contrast to the exceptionally beautiful civic centre. Protracted negotiations, over a number of years, on the financing of the project, the deteriorating economic situation and escalating construction costs, led finally to Ravenseft Properties withdrawing from their partnership with the City. As part of the settlement, Ravenseft paid to the City of Cardiff a substantial sum of money in compensation; this money become the major source of finance for the Concert Hall.

Following the collapse of the partnership, time became the critical factor, so as to prevent the lapse of the compulsory purchase order which was obtained for the land previously earmarked for redevelopment. The City readvertised for developers; however, unlike the first time, where there were over 50 bidders, there were only two serious bidders. Heron Corporation was the organization that was eventually chosen.

Design work commenced in September 1976 following the City Council's approval to the shopping development proposed by Heron. As part of the approval given by the Secretary of State for Wales, the development had to include a building of a civic nature, and it was therefore felt that this location would be ideal for the Concert Hall. This had often been discussed in the past, but had never come to fruition.

From the pure planning point of view, it was felt that a location in the heart of Cardiff would be far better than a green field location in the suburbs as is generally the way with buildings of this nature. Tenders were invited in the early part of 1978 and Laings were successful with a fixed price bid and work started on site in September 1978 following a piling contract undertaken by Piggot Foundations.

The brief

Based upon a Government inquiry into the housing of the arts in Britain, which unanimously recommended that a concert hall be provided in Cardiff, seating has been provided for 2,000. There was to be space on stage for an orchestra of 120 and a choir of 100 with the obvious permutations. In addition, a restaurant was to be provided which could seat 150 people, open to the public all day and yet be an integral part of the Hall together with all the ancillary bar facilities.

The main theme behind the whole design was to be multi-purpose. The auditorium, although its main function was to provide an ideal environment for concerts, had to be capable of providing facilities for conferences, indoor sports, films, etc. The foyer areas were to be available to the public as a general meeting place, for exhibitions and informal small concerts. Backstage facilities had to be in keeping with the front-of-house provisions.

Concept

The design was developed as an integral part of the St. David's shopping centre. The auditorium, which was sited two floors up above the shops, presented a difficult planning and structural problem. The other main constraints were the limited site area and the need to keep the bulk of the building to a minimum to reduce the impact on the street scene. The auditorium was designed to introduce a theatrical element into the Concert Hall and provide an intimacy not normally found in these buildings. The seating is broken into a series of interlinked petal-shaped tiers which wrap around the stage area, thus increasing the rapport between artist and public.

The tiering is arranged symmetrically around the centre line of the auditorium which is at an angle of 22½° to the main shopping grid below.

A major design feature of the auditorium is the stage which is completely flexible. The stage and forestage are divided into various separate hydraulically operated sections.

This enables the stage to be moved to provide whatever facilities the performance may require, such as an orchestra pit, full forestage, orchestra rostra or flat floor.

The front of the house is planned on six levels. Entrance, box office, restaurant and cloakrooms are all on the first three levels, which are interconnected by escalators.

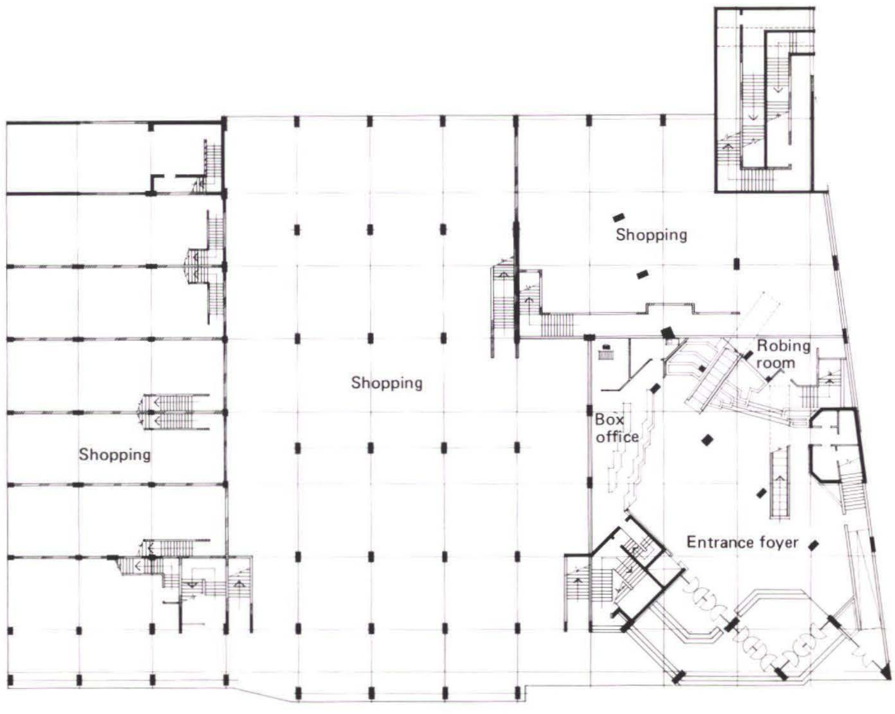


Fig. 1
Ground level plan

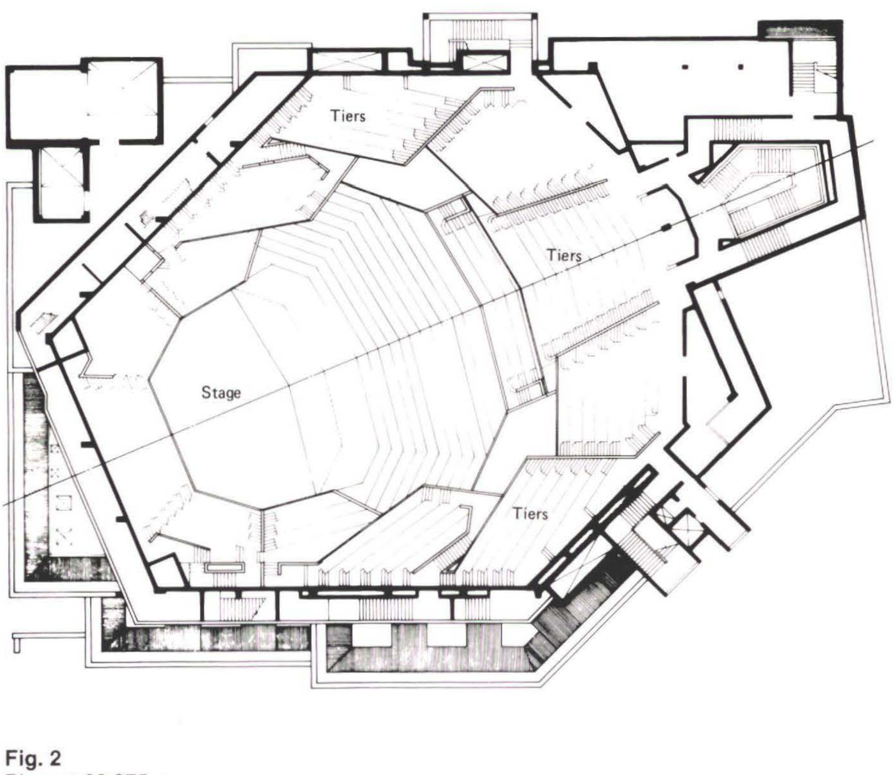


Fig. 2
Plan at 23.975m

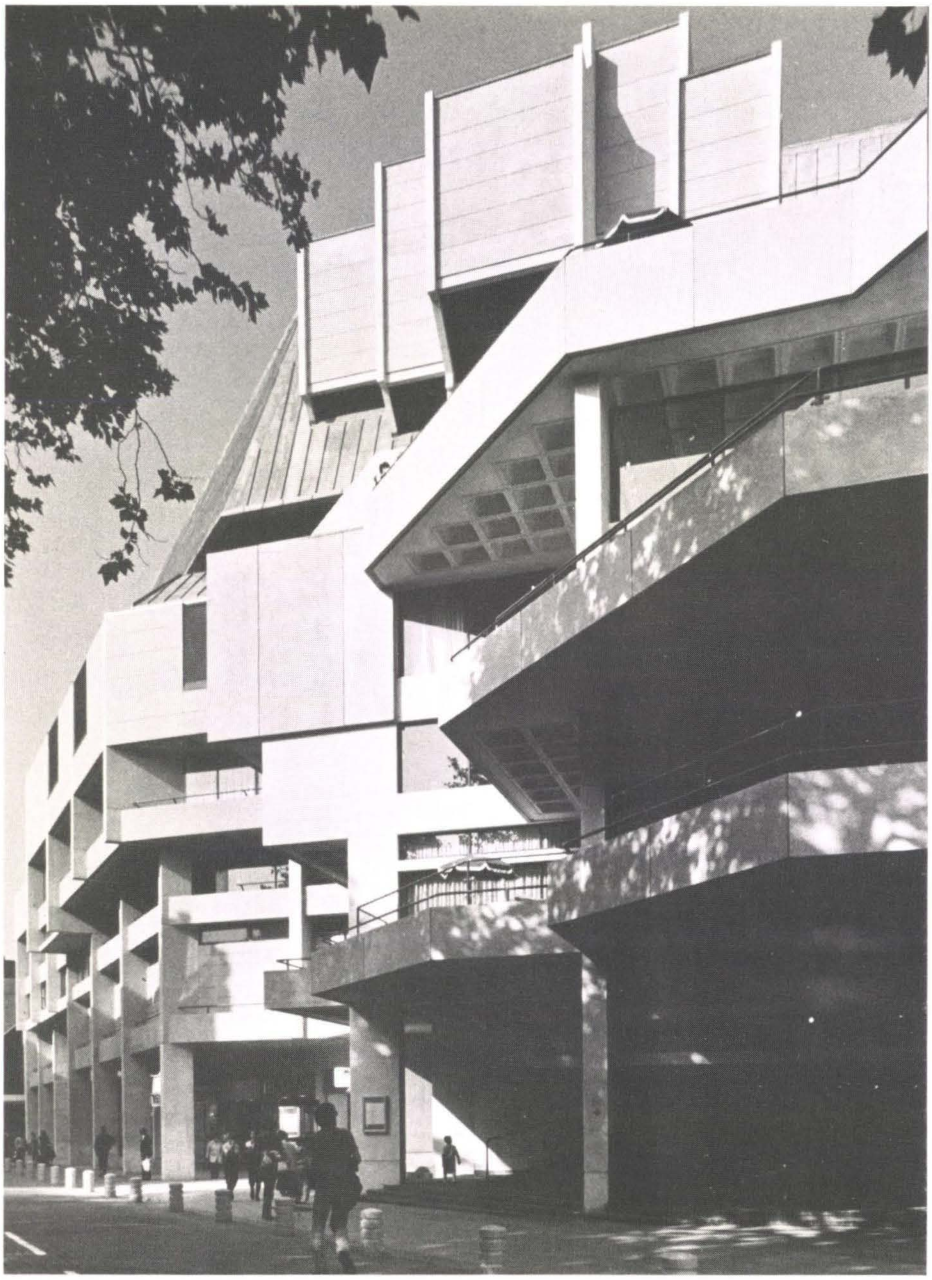


Fig. 3
Elevation of St. David's Hall as viewed from Working Street

Other levels contain bars, foyers, and a coffee shop, which are reached by the main staircase. Terraces and balconies have been created overlooking the street scene at these higher levels.

The internal finishes are designed to be simple and hard wearing. Foyers use the exposed white concrete structure, softened by the grey green carpet. All joinery is finished in ash to an extremely high standard.

Obviously, the acoustic design formed a major part in formulating the auditorium concept. Early acoustic model studies at the University of Cambridge strongly influenced the final geometric shape and finishes of the auditorium. The auditorium design provides a clear speech both natural and amplified, with further extension of the acoustic response, by use of an assisted resonant system, which prolongs reverberation electronically, allowing more scope for music performance.

The suspended ceilings are open cell formalux, which allowed the design to overcome reverberation problems by increasing the volume of the auditorium, without physically increasing the size of the building as a whole.

Structure

The constraints imposed by the site, the need to keep the bulk of the building to a minimum, and the problem of building over two levels of shopping, led to the development of an in situ concrete design with the exception of the large span steel roof.

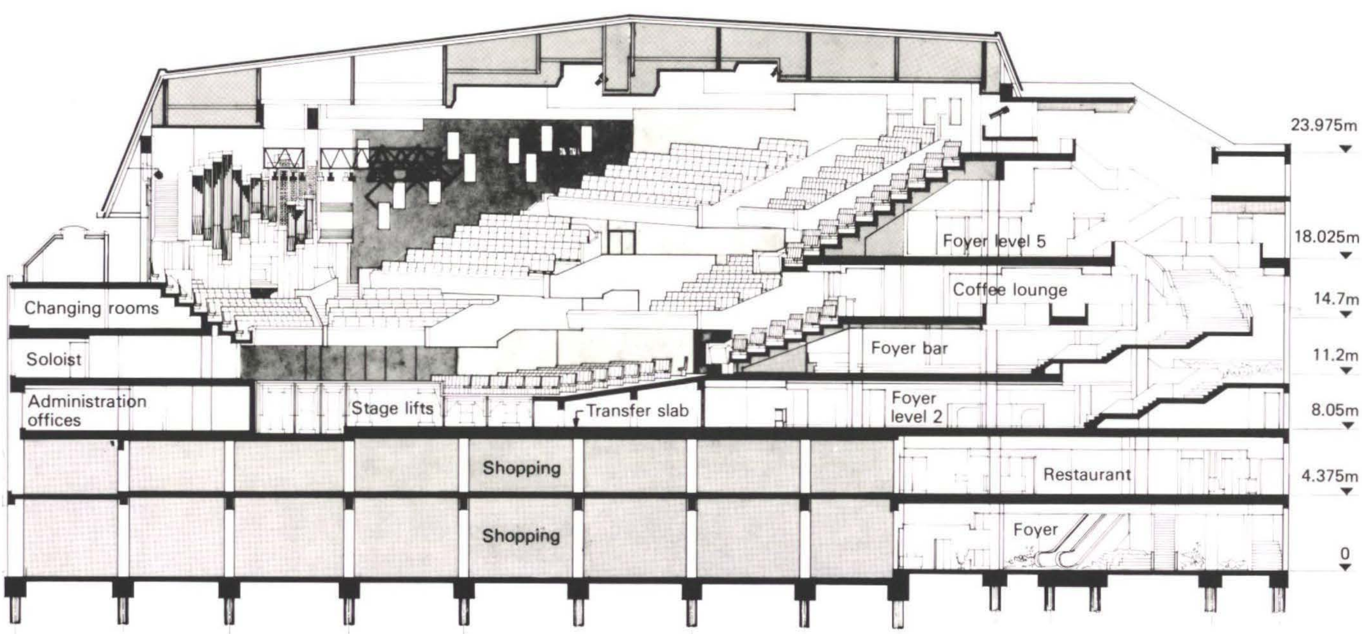


Fig. 4
Cross section

Foundations

Being in the centre of Cardiff, the site was partly occupied by three-storey buildings, the majority of which had basements. Along the eastern site boundary, below ground level, brick and concrete walls followed the line of the old city moat/Glamorgan Canal. Parallel to the course of the moat were the mediaeval town walls, set approximately 7m to the west. To the south of the site stood the SWEB showroom and main electrical sub-station for the City of Cardiff; this building was to be the sole survivor of buildings which adjoined the site.

The general succession of strata on the site, which is fairly typical of Cardiff, was fill or basements to a depth of approximately 2m, a 1.5m band of clay gravel overlying 3.5m of dense river gravel which in turn overlies Keuper Marl at a depth of approximately 7m. Foundations for the Concert Hall are bored cast in situ concrete piles, typically of 2,000 kN to 4,000 kN capacity, 0.7m and 1.2m diameter respectively, and lengths up to 16m.

Prior to the installation of the piles, which were founded in Marl, the site had to be cleared of partially filled basements and foundations that remained after the demolition contract. Guard relays had to be fitted to equipment in the SWEB building so as to prevent equipment tripping out during piling operations.

In view of the thickness of the river gravels, which were overlain by the laminated clays, pad foundations were considered at bearing pressures of up to 400 kN/m^2. However, the differential settlements which were likely to arise mainly where the underlying weathered Marl zones are thick, precluded their use.

Superstructure

The two levels of shopping below the Concert Hall were constructed on the regular grid of 6m x 9m. However, to utilize the maximum space for the auditorium above, the second floor was designed as a 1m thick transfer plate. This allowed the axis of the auditorium to be swung through 22½° to the main shopping grid, and with the exception of the perimeter columns, allowed a degree of freedom for the planning of the tier supports.

Within the auditorium the stall area is formed by casting the in situ floors to falls, the treads and risers being formed in timber. The upper levels are designed as stepped treads and risers, spanning between tier walls which project into the auditorium. The complexity of the form of the tiers, as they fall in two directions, did not allow the contractor to precast treads and risers as he originally intended. In order to obtain a shuttering pattern on the tier walls that was acceptable to the architect, these were constructed first and sockets cast in position to receive reinforcement for the treads and risers; these being cast at a later date.

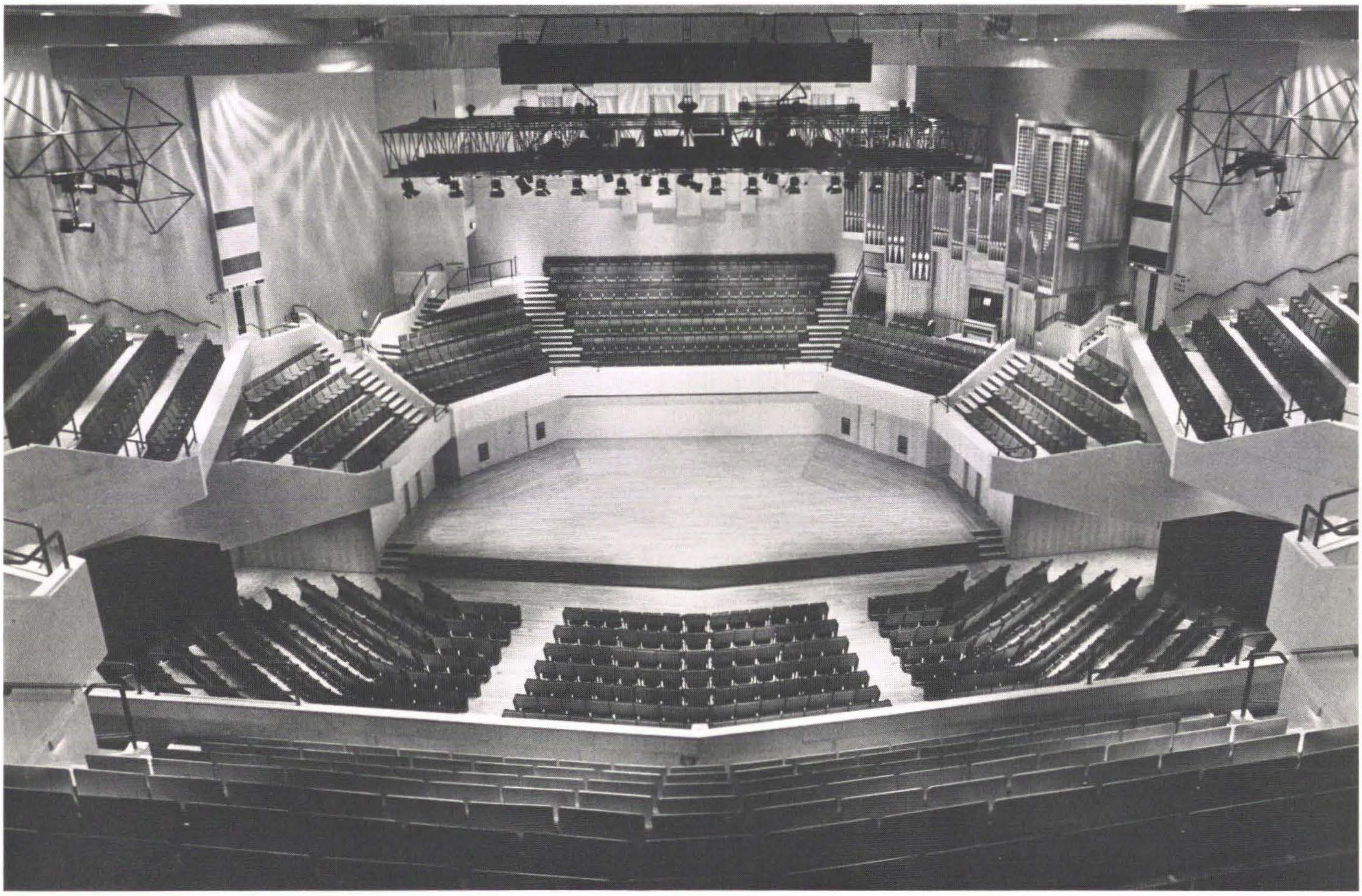


Fig. 5
The auditorium

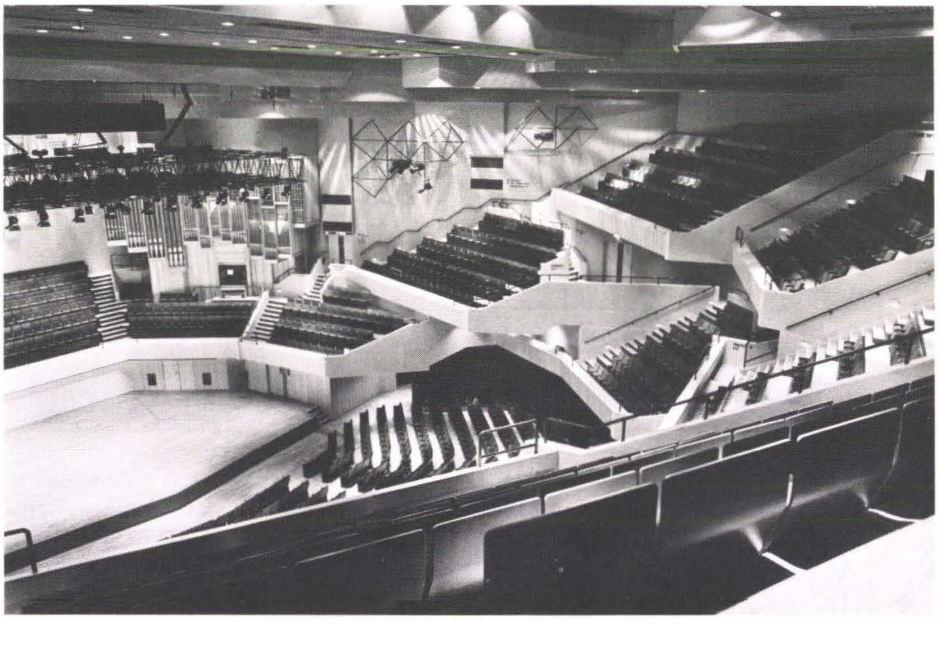


Fig. 6
Side view of auditorium

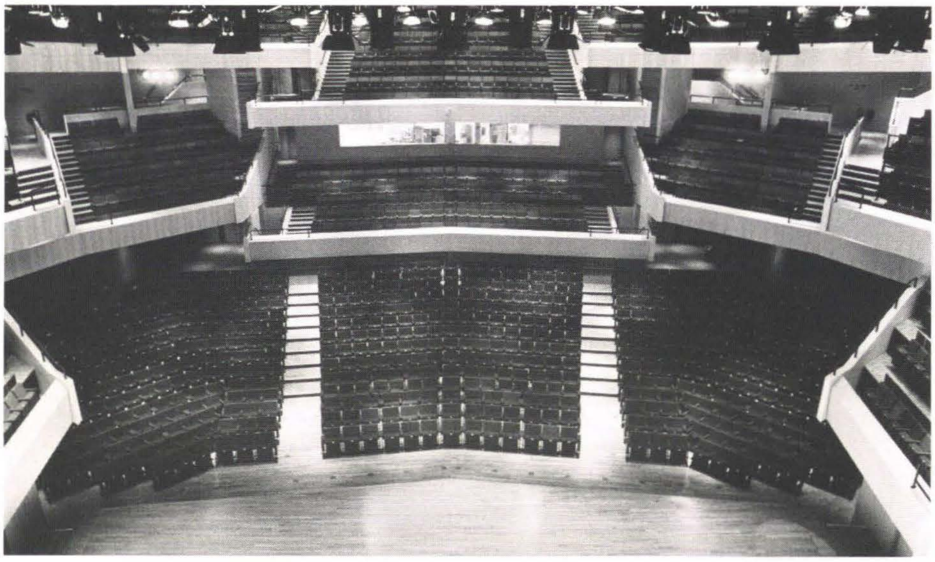


Fig. 7
View of auditorium seating, showing suspended tiers

Fig. 8
Staircases from levels 2 to 3, and above, levels 3 to 4

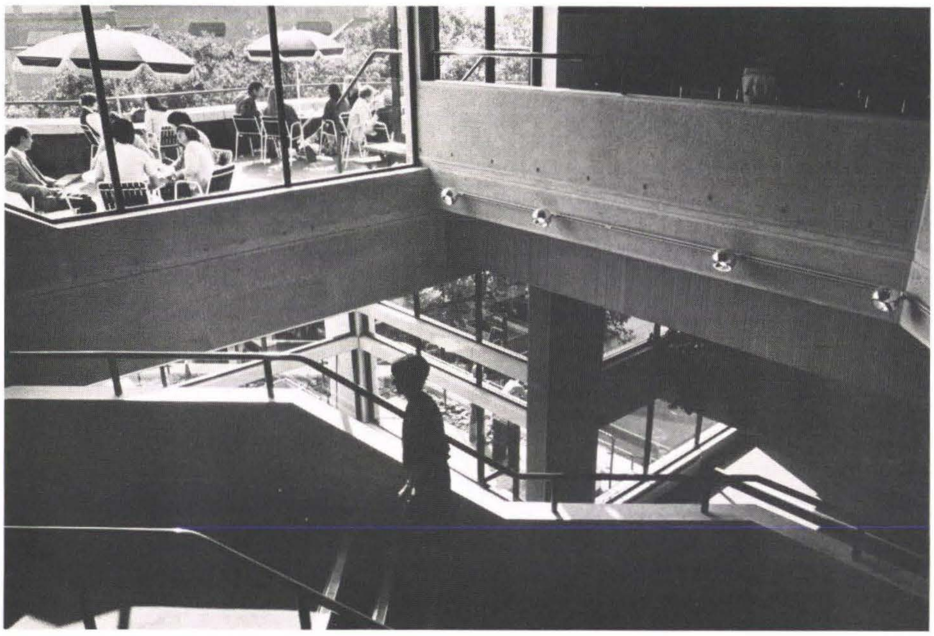


Fig. 9
View from level 4
showing circulation foyer areas and level 4 terrace

In the front-of-house areas the minimum of support has been provided. The slabs have been designed as grillages of flat beams because of the complexity of the floor layout and the need to keep the structural depth to a minimum.

In particular, the level 4 slab is in part hung from level 5. The main staircases span from floor to floor.

To avoid an over-complex roof structure, parallel lattice trusses 3m deep and spanning a maximum of 45m are supported on a peripheral ring beam. The structural depth of the trusses have been kept constant over the entire roof area, falls in the roof being achieved by inclining the ring beam.

Service areas are contained within the structural depth of the roof trusses, and adequate walkways have been provided to service both mechanical equipment and lighting at this level. From the trusses is hung the main lighting grid whose height above stage level can be varied to suit the performers' requirements.

Bearings have been provided beneath the seatings for the truss, limiting movement in the columns and ring beams due to the rotation of the trusses at their point of support.

Bolted site splices at third points enabled trusses to be transported to site in three sections, *Woodcemax* trough sections overlaid by *Woodcelip* slabs spanned from truss to truss which serve as the structural roof deck to the *Broderick* cladding and as an acoustic barrier.

Support for the ring beam is provided in the main by a system of double columns around the perimeter of the auditorium. These columns also provide support and stability to the radiating tier walls. Building stability is provided by concrete stair cores located at the four corners of the building.

The exposed white concrete containing mica, to give a sparkle effect, has been gritblasted with the exception of a margin at the arrises of the column. All white concrete was batched on site so as to provide a high degree of quality control, particularly with regard to colour. All other concrete was batched off site.

Conclusion

St. David's Hall was opened to the public in October 1982 and formally opened on 15 February 1983 by Her Majesty Queen Elizabeth, the Queen Mother. This represented the completion of the first phase of the redevelopment of the City Centre.

The Concert Hall with its conference facilities will enhance Cardiff's standing as the Capital of Wales and provide much-needed amenities in the principality.

Credits

Client:
Council of the City of Cardiff

Architect:
J. Seymour Harris Partnership

M & E engineers:
Sandy Brown MSU

Acoustic consultants:
Sandy Brown Associates

Quantity surveyors:
Cyril Sweett & Partners

Theatre consultants:
Carr and Angier

Main contractor:
John Laing Construction, South West Region

Project manager:
R.G. Flowers

Amersham International plc, Cardiff

Architects: Percy Thomas Partnership

Brian Coles
Gerald Pickin

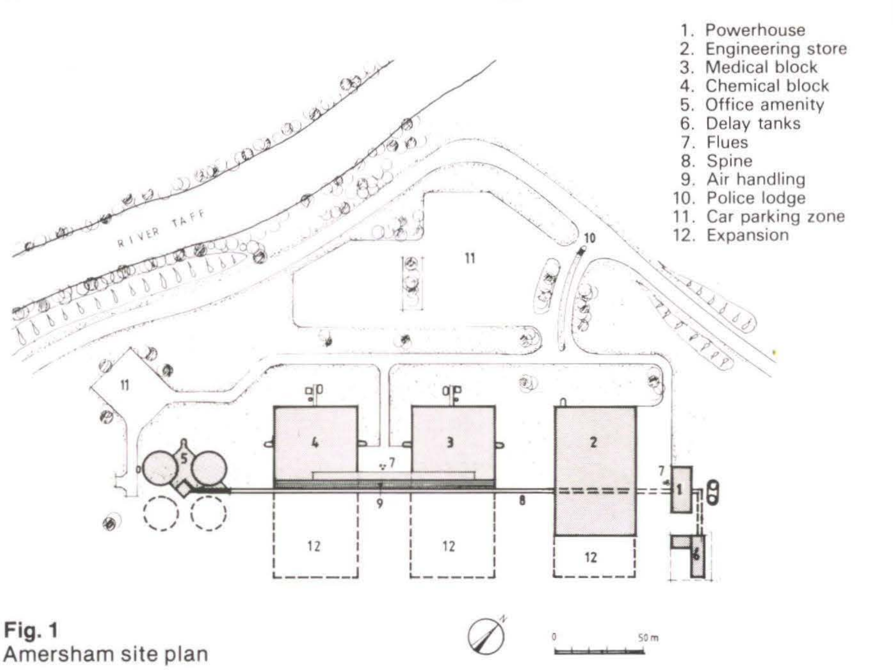


Fig. 1
Amersham site plan

Introduction

In 1974 we were commissioned by Amersham International plc, who were at that time known as the Radiochemical Centre Ltd., to provide structural engineering advice for the construction of a laboratory complex on the outskirts of Cardiff. The complex was required to extend and provide additional production and research facilities to those already present at Amersham. The client spent a great deal of effort in determining the position of the new site, and the particular green field site in Cardiff, which is adjacent to the M4 and within easy reach of Wales Airport, was identified as satisfying the client's needs in terms of area, environment, good communications and availability of quality workforce.

Amersham International carry out production and research work involving the use of radioactive materials. The majority and probably the most important part of their work is the production of diagnostic kits which are used worldwide in medical and industrial fields. Considerable expenditure is committed to research work in developing these kits, particularly in the field of medicine.

Concept

The client's initial requirement was for 2,200m^2 of production/laboratory space, a 3,500m^2 stores and despatch block and the ancillary buildings required to service the complex.

The architect was required to comply with very stringent planning requirements, because of the position of the site and the sensitive nature of the work being carried out. Some of this stemmed from the results of a public inquiry that was carried out on the siting of the development.

The limited shelf life of the products dictated an efficient flow of operations from manufacturing process to despatch. This meant careful integration and layout of each building on the site. The product is such that, particularly in the field of medical diagnosis, the time between order and use can be measured in days.

The nature of the work necessitated complex and sophisticated servicing, particularly to the production and laboratory block. A major design consideration therefore evolved around the air extract system to these blocks. Contaminated air is drawn to the air handling block where it is filtered and monitored before being expelled through the 50m high GRP flues. Similarly contaminated effluent from laboratory benches is taken in doubly contained waste pipes to the air handling block for collection and monitoring, prior to pumping into holding tanks for disposal.

Fig. 1 shows the layout of the buildings on the site. The spine of the complex is an underground duct and link corridor, which services and provides access to all buildings on the site.

The architect adopted a low rise approach to the design of the complex, and chose to clad the larger buildings with GRP, seeking to soften the elevations of these buildings, but maintaining the high-tech image of the client. The power house and office amenity elevations were clad entirely in reflective glazing.

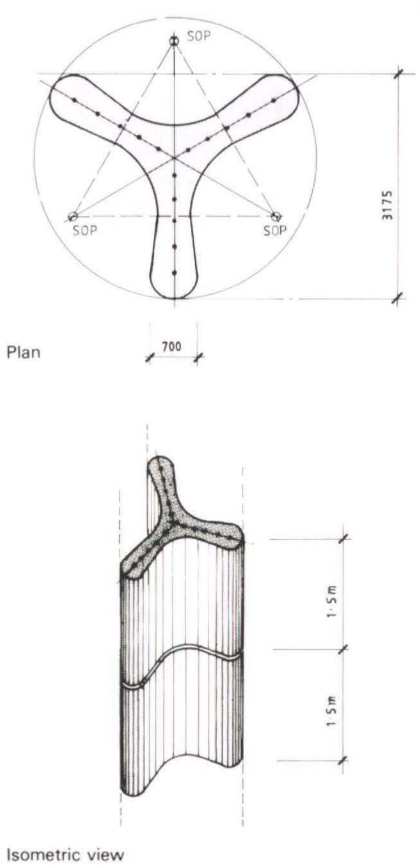


Fig. 2
The air handling stack

Design

The site is underlain by the River Terrace Gravels which allowed the use of simple pad foundations throughout the scheme. The water table was approximately 4m below ground level, generally allowing formation levels to be excavated above the water table.

The power house and engineering stores block were both single storey structures utilizing a structural steel frame. The two-storey production blocks were of conventional in situ reinforced concrete construction utilizing waffle moulds. Following the removal of formwork, the concrete soffit was sealed to prevent dusting which would contaminate the ceiling service void.

It was essential that all concrete was sound and only very minor defects were permitted. A high standard of construction was maintained throughout the complex. The production blocks are linked and serviced by the air handling block, which is a three-storey structure, again of reinforced concrete construction. This block also has a two tier basement, the lower level being some 2m below the water table. Water-retaining concrete was used for the construction of this section of basement. The two circular office and amenity blocks are of two storeys and reinforced concrete construction; the structure comprises a central circular concrete service core with perimeter columns supporting concrete flat slabs. Water retaining concrete construction was again used for the linking spine service duct. The access corridor above is structural steelwork. The spine duct terminates at the effluent holding tanks which are constructed of water retaining reinforced concrete. The holding tanks consist of three tanks within one large tank maintaining the double containment principle.

The most prominent features of the site are the two chimneys, with the 50m high air handling stack dominant. Both stacks comprised three flues which the architect wished to express rather than contain them within an external wind shield. The air handling flues were constructed of GRP, and the boiler house flues were of stainless steel clad in GRP. Several steel and concrete schemes were considered for the construction of the supporting pylon. Detailed consideration was given to the shape of the supporting structure and a trefoil configuration was derived by developing the space bounded by the three circular flues. The shape is shown in Fig. 2.

Having decided upon the shape, finish and construction methods were considered. A high quality finish was essential which ruled out a possible slip forming construction method or any other in situ method. It was decided that precast concrete would provide the optimum solution. The match face casting technique was used to make the precast units. The concrete used in the units was made from a white ballidon aggregate

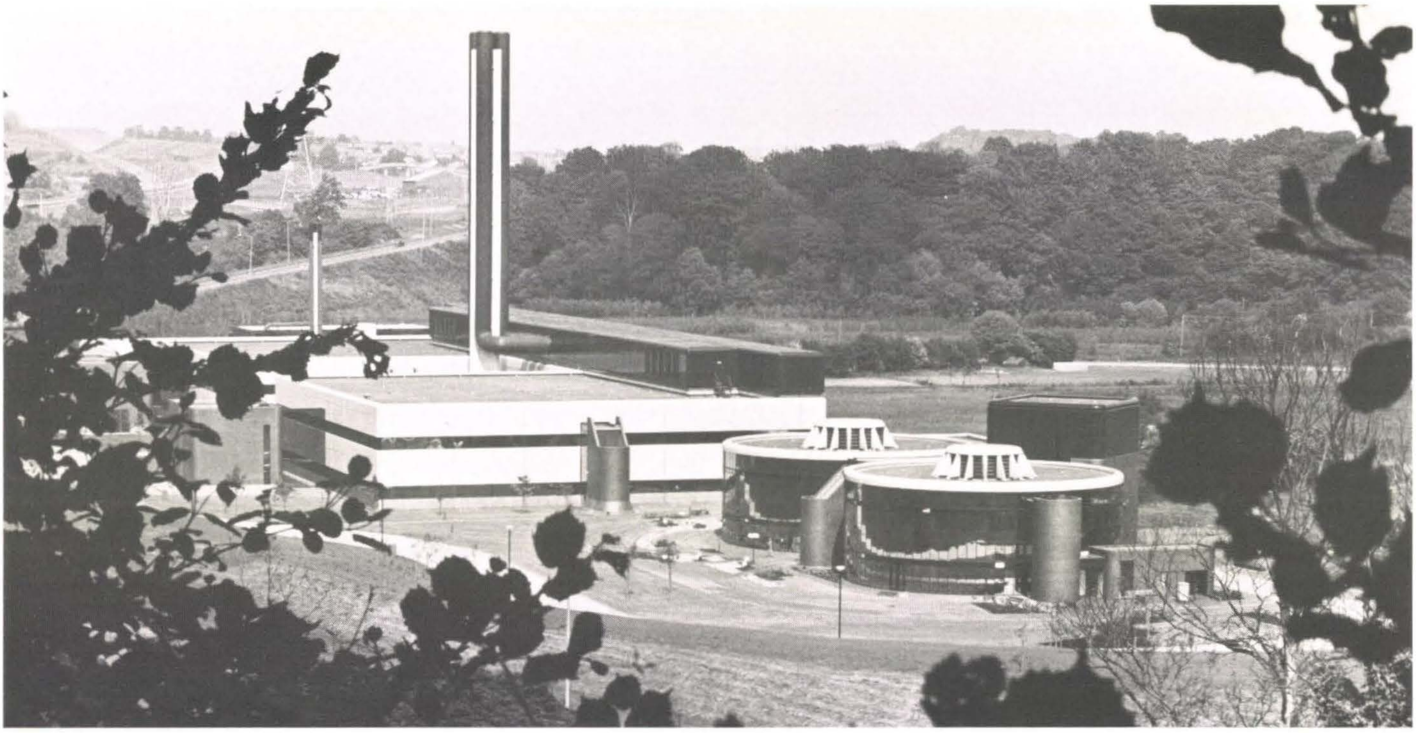
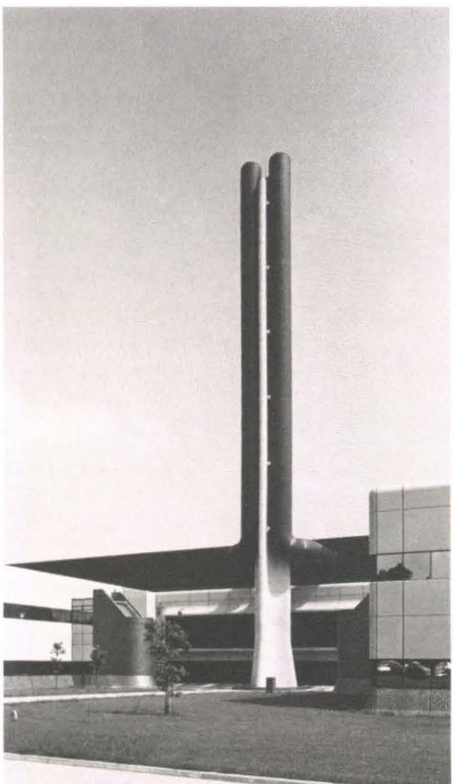


Fig. 3
General view from the River Taff

Fig. 4
The 50m air handling stack

Fig. 5
The office and amenity blocks



together with white cement and vertical surfaces of the units were given a uniform, textured finish by bush hammering. The two supporting pylons were then designed as post-tensioned structures with each joint being glued together with an epoxy resin. The 25m high boilerhouse pylon was post-tensioned using nine Macalloy bars and these were stressed at three levels in the height of the structure.

Post-tensioning of the 50m high air handling pylon was achieved by 15 prestress strands which were drawn through the sides of the foundation and sequentially stressed at five levels through the height of the pylon. The boilerhouse stack was constructed first and proved the design by being completed within 10 days; maximum out of plumb was 10mm. There were some problems experienced with the plumb of the air handling stack during construction. Corrective measures were necessary using glass fibre shims bedded in the epoxy resin.

Construction

Construction work began in 1976 with an initial contract comprising the engineer stores and despatch block, a power house, the delay tanks, and some peripheral storage buildings. The main contractor for this phase was R.M. Douglas Ltd. Construction of the complex was phased to allow the technically simpler buildings to be erected, allowing further design to continue on the more complex production blocks. It also allowed an initial move of training personnel from Amersham. The engineering block was later used to store laboratory benches prior to installation in the next phase. Phase 2, comprising the remainder of the initial scheme, started in 1977 with Tarmac National Construction Ltd. as the main contractor. The second phase was completed by 1980. A third phase is planned which will give additional production, laboratory, office, and storage accommodation.

Awards

The design of the complex has been very favourably received and has been given awards by the Financial Times, the RIBA, The Concrete Society and the Royal National Eistedfodd of Wales.

Credits

Client:
Amersham International plc

Architect:
Percy Thomas Partnership

Quantity surveyor:
Patterson & Seaton

Services engineers:
The Atomic Energy Research Establishment

GRP consultants:
Peter Hodge and Associates

Photos:
Harry Sowden

New factory at Abercanaid

Consulting Architect:
Percy Thomas Partnership

Dick Hensby

Fig. 1
The site before development (Photo: Terence Soames)

Introduction
Ove Arup and Partners were commissioned by the Welsh Development Agency in late 1973 to act as their principal agent and structural and civil engineering consultant for a new factory complex on the west bank of the River Taff, just south of Merthyr Tydfil. In the following year, the commission was extended to include the design of the mechanical and electrical services.

The factory was designed for use by Hoover Ltd. to cater for a planned expansion of their operations at Merthyr Tydfil.

Following site preparation contracts in 1977 and early 1978, the main construction contracts proceeded in the period spring 1978 to autumn 1980, with handover to the client being achieved in October 1980.

Unfortunately, a downturn in demand has since resulted in a general contraction of Hoover Ltd. production in the UK, and the complex is now being actively marketed by the Welsh Development Agency under the new name of 'Dragonparc, South Wales'. In this regard, the design philosophy of providing flexibility for future changes in use is now seen to particular advantage.

History
Hoover Ltd. established a factory in Merthyr Tydfil immediately after the Second World War and, following a steady expansion of their production of home laundry products, had by the early 1970s, fully exploited their east bank site covering an area of 24ha and employing over 5000 people.

Market trends at that time indicated that a large new factory was called for to meet an increasing demand, and a site was identified in 1973 adjacent to the existing factory, but on the far side of the River Taff. The attraction of building in the close proximity of the old factory and thereby rationalizing production and administrative functions outweighed the not inconsiderable problems of preparing the site.

Quite apart from the obvious physical obstructions, it was also necessary to arrange for the re-routing of the proposed Cardiff to Merthyr Tydfil trunk road from the preferred alignment by agreement with the Welsh Office.

Development of the brief
Initially, the brief called for a two-storey building, modelled on an existing Hoover Ltd. factory in the United States and indeed, the original site was only large enough to accommodate the required floor area if two storeys were adopted.

Ove Arup and Partners then suggested that if further land could be acquired, it would be possible to achieve a single-storey building, subject to agreement to realign the local village access road. This would lead to reduced overall cost and enhance the flexibility of the completed factory.

This proposal was adopted and the road rerouted and designed to incorporate roundabouts for access to the factory to deal with the anticipated traffic flows.

At this stage, Hoover Ltd. indicated that they would contain all of their requirements within the factory envelope. Ove Arup and Partners, in conjunction with the consulting architect, observed that this would lead to a very inflexible use of the building and suggested that all of the ancillary functions should be drawn outside the main building. Here, they could be extended and adapted as necessary to meet changing requirements in the years ahead without reducing production space.

The final brief to the design team called for the design of an industrial complex comprising:

(a) A single-storey production building of 25,000m² floor area, with provision for expansion to 42,000m², together with various annexed buildings and structures to accommodate ancillary services and welfare facilities associated with production.

(b) A multi-storey office/production engineering and test building having a usable floor area of 4,400m², with provision for a basement car park to accommodate approximately 60 vehicles.

(c) Canteen and kitchen facilities having a usable floor aea of 1,800m².

(d) Provision to be made for a total of 650 car parking spaces, plus lorry marshalling and waiting areas.

(e) A pedestrian bridge linking the office building to the existing Hoover complex.

Design
Site preparation
The site covers an area of 14 ha once partially occupied by tips of colliery, iron foundry and domestic wastes, rising to 20m above the final site level and which had remained generally undisturbed since they were laid down at the end of the last century.

A site investigation contract was let in 1974, to establish the quality of the tip materials and of the glacial materials beneath and to determine the geology of the underlying coal measures. This was followed by other investigations to check the compaction and combustibility characteristics of the tip materials.

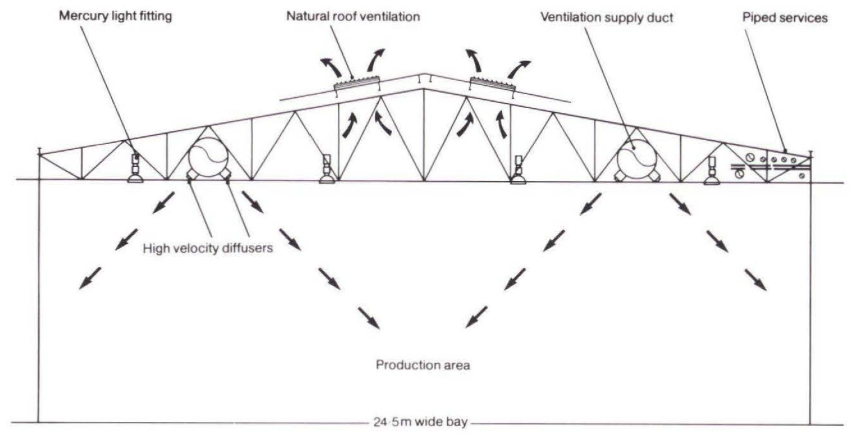
Preliminary site preparation was carried out under contracts let by the Gwent Land Reclamation Joint Committee and K Wardell and Partners, to remove surplus tip materials, divert roads and services and to drill and grout the unstable zones in the old coal measures below the site.

Ove Arup and Partners then designed and supervised river training and site compaction contracts to provide a sound building platform secure from flooding or erosion, and to maximize the area available for development. A permissible ground bearing pressure of 250kN/m² to accept unreinforced concrete floor slabs and column loads up to 2000kN on single pad footings were the design criteria.

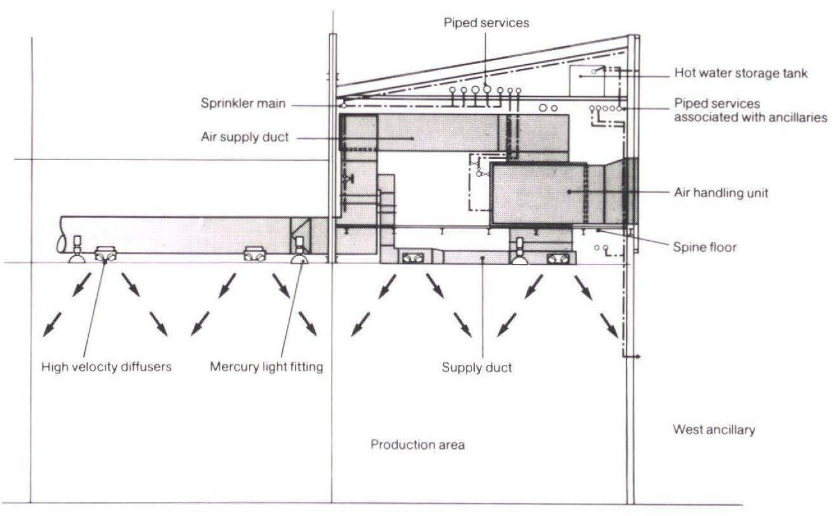
The river training work, carried out in 1977, called for the design and construction of a 500m long reinforced concrete retaining wall surmounted by selected stone fill at a 45° slope. This site protection safely withstood the effects of a flood of over 1 in 100 year return period in December 1980, which caused extensive flooding and damage elsewhere in the valley of the River Taff.

The site compaction contract required the excavation and re-compaction, to a depth of 2.5m below formation level, of 250,000m³ of colliery spoil and the removal from site of all unsuitable material. This work was carried

Fig. 2
Typical view inside the production block (Photo: Terence Soames)



Typical roof bay



Typical spine plant room

Fig. 3
Production block: mechanical services

out successfully during the winter of 1977/78, and owed much to careful and thorough site supervision throughout the contract period.

An alternative form of site treatment by dynamic compaction had been previously considered, but ruled out on cost grounds.

The way was then clear to proceed with the main construction contracts in spring 1978.

Elevational treatment
Due to the varied forms of the component blocks of the development, from the massive production block to the small security lodges, a strong unifying factor was needed throughout the whole site. This has been achieved by the use of only two basic materials, namely brick and pressed metal sheeting on the upper section of each block. The two materials are separated by a recessed pressed metal band which is a second unifying factor, as it helps to locate one building against another and to unify various items of strong elevational expression such as doors and windows. By limiting the height of the brickwork, the band also helps to provide a more acceptable human scale on the larger blocks.

Landscaping has been carried out to assist the integration of buildings with the surrounding areas for the benefit of both factory personnel and the off-site observer.

Structural form
The main feature of the complex is the production block building. This has a steel frame construction to permit maximum adaptability of use. This theme is reflected in the engineering services design and is fundamental to the manufacturing requirements for the consumer durables market where changes in demand for a product may lead to an urgent need to strip out and replace production line equipment.

Main roof members are of lattice truss construction and accommodate the mechanical and electrical services distribution within the roof space. In addition, the trusses will also support an overhead conveyor installation up to a load of 1.5kN/m^2.

There is a clear height to the underside of the roof trusses of 7.65m to meet the special requirements of such a conveyor system.

The steelwork is protected by the *Metalife* corrosion protection paint system and columns in external walls are generally concrete-cased.

Elevated spines, or galleries, have been introduced along the outer bays for the full length of the building to accommodate the heating and ventilating plant, the 11kV ring main equipment and the mechanical and electrical services distribution. This provision maximizes the area of uncluttered production floor space and facilitates the routine maintenance of the equipment. Within the spines, the precast concrete deck platforms and the external cladding are readily demountable to afford access for equipment replacement.

Eight steel-framed, mainly single-storey ancillary buildings are provided around the perimeter of the production block to accommodate items of process and maintenance plant and also welfare facilities.

The cafeteria/boilerhouse complex is principally a single-storey steel framed structure of lattice girder and column construction. Girders are all 12m span at generally 4m centres, but are increased in the boiler house area to suit the boiler dimension. The two mezzanine plantroom floors are of steel frame and precast concrete unit construction, suspended from the main roof members. Steelwork corrosion protection is similar to the production block.

The office block is a four-storey reinforced concrete frame structure with columns on a 6m grid with a central light well to the upper two floors affording maximum daylight.

The 500-space, double-storey car park is constructed of in situ concrete columns on a 15.5m × 4.6m grid, supporting precast, prestressed, concrete bridge beams. The beams in turn, support concrete planks with an in situ concrete topping and waterproof wearing surface.

Environmental and process services
Three 5861kW high pressure, hot water, dual-fuel (oil/gas) boilers provide the demands of process, heating and hot water requirements. Sufficient space has been allocated in the boiler house to accommodate a further boiler of similar capacity to meet any additional load from the possible Production Block extension.

Heating and ventilation in the production block is generally provided by a two-speed, all-air ventilation system capable of supplying three air changes per hour minimum fresh air during winter, with an increased ventilation rate of six air changes per hour for operation during the summer period.

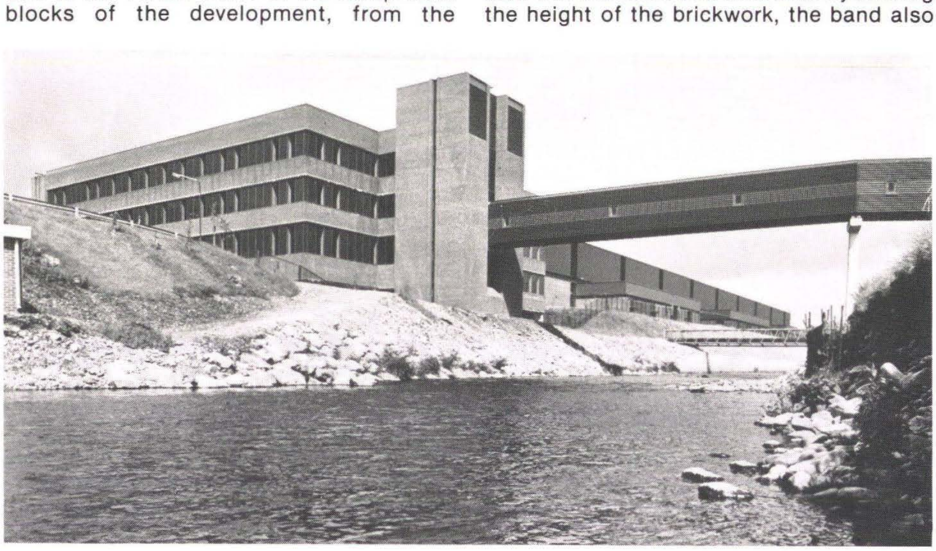
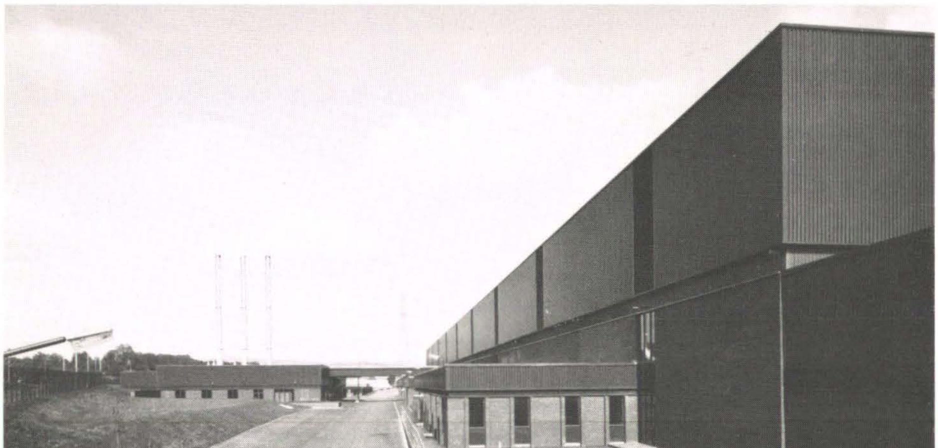


Fig. 4
Bridge Link: Office block to the original Hoover complex (Photo: Harry Sowden)



Figs. 5-7
Views of the Abercanaid Factory (Photo: Terence Soames)

Each 24.5m bay is served by two air-handling units located in the service spines, with distribution from either side of the factory. In each air-handling unit, control of leaving air temperature is by a remote thermostat located within the distribution pattern of the unit.

The 18 air-handling units are supplemented, where noxious or more demanding processes require, by supply air units installed within the roof void.

Cold air curtains are provided at the larger openings to minimize infiltration of external air within the production building. Other services comprise high pressure hot water for both process and space heating, cooling water, compressed air, fire mains, drinking water, natural gas and sprinkler protection.

The high voltage supply has been obtained by splitting and extending the 11,000 volt ring system in the original east bank factory site. The cables are carried over the intervening railway and river by overhead conductors on poles on the north crossing and via the pedestrian link bridge to the south.

The six transformers serving the production block medium voltage distribution, are located in the service spines and are grouped into three pairs interconnected across the production block at 7.65m above finished floor level by triple pole and neutral 1700 ampere busbar trunking.

To cater for the flexibility required, the electrical services design provides a blanket coverage to all areas. Lighting design levels are 600 lux and power distribution has been designed generally to allow a maximum supply of 60 amperes triple pole and neutral, to be taken from the supply system at frequent intervals using a busbar system for ease of connection and disconnection.

In the office block, the ground floor accommodation is a mixture of engineering workshop and laboratory activities which require a high level of services for their functions.

The upper floors consist mainly of large, open plan administration and drawing offices, together with some cellular offices for managers and secretaries. These floors have been designed in detail for future adaptability of layout. This adaptability has been achieved by the provision of a ceiling system installed on a 1m grid, which allows location of demountable partitions at 1m intervals in both directions, giving a great degree of flexibility for future planning changes. The building services design also reflects this approach in the provision of ventilation air grilles, power points for electricity and also telephone connections at appropriate intervals.

A balance has been achieved between the degree of adaptability of accommodation and the problems of glare and solar gain by providing a pattern of narrow vertical windows placed at regular intervals for good vision to the outside, and which also aids environmental control of the office spaces.

Toilet facilities, ancillary plant rooms and a six-person passenger lift are provided in the separate fully-enclosed staircase towers. This arrangement further enhances the potential for the office space to be planned for maximum adaptability.

Sprinkler fire protection is provided to the production area including the service spines and the ancillary buildings. Cut-off sprinklers are installed over the entrance to the link bridge between the production and office blocks. In addition, sprinkler alarms are located in the spine sub-stations.

The sprinkler installation is designed for Extra Hazard Category III protection to the production area, service spines, underside of plant platform, the paint mix and inflammable stores. Protection elsewhere is to ordinary Hazard Group III. The Extra High Hazard protection provided permits a maximum storage height of 5.2m in Category III.

An unusual feature of the services design is the arrangement with the Welsh Water Authority to abstract 400,000 gallons of non-potable water per day from the outlet of a culvert on the south side of the site for use, after treatment, for process purposes and in WCs up to a maximum of 112 million gallons per year. This quantity of water has been assessed against predicted demand in both the old and new factory complexes and also takes into account the dry weather flow potential of this source of water, which in drought conditions, is derived solely from a disused NCB mineshaft located to the west of the site. This system leads to significant savings in the factory operating costs.

Credits

Client:
Welsh Development Agency

Principal agent:
Ove Arup & Partners

Consulting architect:
Percy Thomas Partnership

Quantity surveyor:
I.E. Symonds & Partners

M & E Consultants:
D.G. Batley, Controller (Engineering Services) Welsh Development Agency, in association with
Ove Arup & Partners (Manchester)

Main contractors:
(River Training):
Andrew Scott Ltd. (Civil Engineers)
(Site earthworks):
Shephard Hill & Company Ltd.

Contract A:
(Production building, cafeteria/boiler house and external works):
Wimpey Construction (UK) Ltd.
Contract B:
(Office block):
G. Percy Trentham Ltd.
Contract C:
(Double storey car park):
Wimpey Construction (UK) Ltd.

Structural steelwork sub-contractor:
Contract A:
Redpath Engineering Ltd.

M & E sub-contractors:
Contract A:
Drake & Scull Engineering Ltd. (M & E)
Matthew Hall & Company Ltd. (Sprinklers)
Contract B:
J.C. Hitt Ltd. (Mechanical)
Haden Young (Electrical)
Contract C:
Drake & Scull Engineering Ltd. (Electrical)

Site Investigation
Nuttall Geotechnical Services Ltd.
Soil Mechanics Ltd.

Proposed hydrodynamic complex at Glasgow University

Bob McLeod
Tom Ridley

Preface

As offshore mineral exploration and production moves into deeper waters beyond the continental shelf, e.g. west of Shetland, many innovative concepts for platform development are being evolved.

Most of these concepts involve slender members for which both the imposed hydrodynamic loading and response are not amenable to straightforward mathematical analysis. The consequences of prototype structural failure are usually so substantial that considerable effort is expended to ensure that the operational behaviour is predicted with the greatest degree of confidence. Despite considerable recent advances in computer analysis techniques, hydraulic model testing remains a requirement for the successful design and certification of most offshore structures.

Many existing fixed offshore structures such as concrete gravity platforms where hydrodynamic loading is predominantly inertial can be successfully modelled at relatively low scale in existing wave tanks at scales of 1:100. Greater uncertainty persists in the design of more slender compliant structures with lower member diameter/wavelength ratios where Reynold's number effects become significant. If the model is in the subcritical regime, not only does the drag coefficient differ but the oscillatory nature of the flow can produce unpredictable vorticity effects.

The consequence is that for offshore structures with relatively small diameter members, the scale should be sufficiently large to ensure that the wave flow regime is the same in model and prototype (usually supercritical). For typical North Sea offshore environments this requirement can necessitate a very large hydrodynamic test facility such as that described here.

Introduction

Ove Arup and Partners were asked by the University of Glasgow to undertake a feasibility study of the design and construction of such a high-wave, deep-water hydrodynamic test facility adjacent to their existing laboratory at Acre Road, Glasgow. The University's Department of Naval Architecture and Ocean Engineering this year celebrates its centenary. It has an international reputation for research encouraged by a long association with shipbuilding and marine-related industry on the Clyde. The facility described here looks towards the future of offshore mineral exploration in the hostile waters beyond the continental shelf and the development of the essential scientific data for the new technologies required.

The implementation of a design fulfilling all aspects of the original ambitious brief specified by the University was found to cost more than the budget initial capital cost of £3 million.

Considerable effort was subsequently expended in endeavouring to identify several areas where significant cost savings could be achieved by reducing the requirements of the original specification without prejudicing the functional effectiveness of the new facility. Non-essential items such as current, wind and workshops were then removed from the initial capital cost, but careful thought given in the design to facilitate their later provision. The study concluded that it was indeed possible to satisfy the essential features of the specification at around the overall budget cost and made recommendations for further studies that should proceed immediately if the target commissioning date of October 1985 was to be achieved.

The design that evolved has been discussed at some length by interested bodies and it is widely felt that it offers a significant advance in wave height and water depth over any existing experiment tank, including the new deep water tank at Trondheim. In the ocean engineering field it will have maximum benefit for tests on jacket structures, tension leg platforms, articulated columns and other compliant structures, but will also give significantly improved conditions for tests on semi-submersibles, buoys, etc. For the first time it will be possible to simulate full-scale flow conditions on models of structures and obtain the correct vortex shedding and interference effects. This will enable much more accurate full-scale predictions to be made. The larger models will enable dynamic characteristics to be much more closely modelled and allow reliable predictions of undesirable oscillations of cables, risers, etc. From the research point of view it will allow engineers the opportunity to study the detailed flow mechanisms around structures in waves that will reproduce the conditions encountered on the full scale in the North Sea.

Scale

As one attempts to locate platforms in deeper and deeper water, either by stretching existing design concepts or by the introduction of new ones, dynamic problems become of increasing importance. The modern concepts of tethered buoyant platforms and guyed towers are dependent on satisfactory behaviour of their moorings. To model these systems satisfactorily it is necessary not only to reproduce the fluid flow, but all the stiffness and damping characteristics of the cables should also be correctly modelled. In order to overcome these problems, one has to test large scale models in appropriate wave conditions. For most platform systems the desirable scale is around 1/15 to 1/20 but a larger scale is

Table 1: Tank dimensions as proposed by Glasgow compared with major world tanks having water depth of over 3m.

Country	Tank	Length × breadth m × m	Water depth: Overall m	Pit m	Wave characteristics: Height max. m	Multi directional
	GLASGOW PROPOSAL	65 × 18	10.0	10.0	1.5-2.0	Yes*
Europe:	Denmark	240 × 12	5.5	No	0.40	No
	Paris	220 × 13	4.5	No	0.53	No
	Hamburg	300 × 18	5.7	No	0.45	No
	De Voorst, Delft	233 × 5	9.5	No	2.5	No
	Trondheim No. 1 & 3	260 × 10.5	5.6 & 10	No	0.9	No
	Trondheim Ocean Laboratory	80 × 53	5-10	No	0.5 0.9	Yes
	Swedish Maritime Research Centre Basin	89 × 89	0-3.5	Yes	0.4	Yes
	National Maritime Institute No. 3 Tank	400 × 14.6	7.6	No	0.5	No
	Admiralty Marine Testing Establishment No. 2 Tank	271 × 12.2	5.5	No	0.6	No
	Admiralty Marine Testing Establishment Basin	122 × 61	5.5	No	0.6	Yes
Japan:	Ship Research Institute	400 × 18	8.0	No	0.45	No
	I H Marine Co.	70 × 30	3.0	No	0.4	Yes
	Nippon Kaiji Kyokai	240 × 18	8.0	No	0.52	No
	Nagasaki	160 × 30	3.5	No	0.40	Yes
USA:	Offshore Technology Centre, California	89 × 15	4.5	9.0	0.75	No
	Chicago Bridge	76 × 10	5.5	No	—	No
	Naval Academy	128 × 7.92	4.87	No	1.0	No
	Hydronautics	127 × 7.6	4.0	9.1	0.6	No
Canada:	National Research Council	122 × 61	3.7	No	0.6	Yes

*Not cross waves

needed for specific elements such as risers or moorings. The model scale is attempting to reproduce extreme sea conditions where the waves may be of the order of 450m long and 30m high, and depths of water applicable to fields for development in the near future are in the range 100 to 400m. At 1/20th scale, the model wave lengths must be up to 22.5m with heights up to 1.5m and the water depth up to 20m deep. The disturbance due to waves dies away comparatively rapidly below the surface, but to reduce any distortion of the waves due to shallow water effects to a barely acceptable level it would be desirable to have a water depth of at least one sixth of the wave length, i.e. greater than 5m. In practice the most important consideration governing depth is the necessity of modelling moorings correctly, e.g. if one is modelling a tethered buoyant platform in 400m of water the model tethers at 1/20 scale must be 20m long. Gross distortion of the depth scale will be unacceptable.

Essential features of the study brief

The above considerations dominated the technical design of the tank proposed at Glasgow University. It was felt essential to make a major step change in experimental facilities which could greatly increase confidence and hopefully reduce costs in the exploitation of deeper waters. Table 1 shows a comparison of the Glasgow proposed dimensions with the major world tanks having a depth of water greater than 3m.

From the table it will be seen that the significant differences in the Glasgow proposal are the wave height and the tank depth.

Apart from the De Voorst Laboratory, which has only a narrow tank unsuited to offshore structures testing, no other laboratory has the ability to simulate the flow conditions experienced by the full-scale structures.

Waves

Ideally an experiment tank should reproduce the complete environment as closely as possible, including currents and winds. Real seaways will have a considerable directional spread and may have cross waves when one storm has followed another, but from a substantially different direction. Thus, ideally, an experiment tank should have a capability of producing waves along two sides at right angles and on each side provide for some directional spread. The study brief called for the provision of regular waves having maximum steepness up to 15m in length, and a height of 1.5m from lengths of 15 to 30m from a single direction with a directional spread of up to 18°. Allowance was also to be made for further wavemakers to be fitted on one of the longer sides of the tank at some future date.

Reflected waves and standing waves arising from wave generation or from the models or tank walls are a major problem in experiment tanks, especially if it is desired to generate random and directional seaways. The brief called for the absorption of such waves in the longitudinal direction by active wave absorbers.

Current

The highest current likely to occur on the full scale is of the order of 2 m/s and on a 1/20th scale this would indicate a speed of 0.5 m/s. Such a current might occur in any direction relative to the waves. The design study called for an examination of the cost of providing this level of current across the model test area which was defined as being 8m wide over the full depth including the pit.

Wind

Wind conditions are very difficult to simulate accurately over a wide area and thus the study brief called for a wind speed of at least 8 m/s, equivalent to 25 m/s on the full scale, to be provided locally, close to the model test area.

Operation

Since the type of tank proposed would involve high daily running costs it is important that the turn-round on test programmes be as swift as possible with little or no delay between different models.This is a further factor demanding quick wave absorption to cut down the time between individual test runs. Also the means of handling and installing the models must be efficient and, if possible, allow for one model being instrumented while the previous test was running.

Cost

The problem of capital cost was a further dominating factor. A previous study for a National Tank Facility[3] was described in *The Arup Journal* for December 1980. It had shown that the capital cost for the ideal solution was greater than £30m. which was beyond what industry or government were willing to fund. It was believed that there was a substantial body of engineers who recognized the need for improved facilities, but the University's assessment of the financial backing that might be realizable was £3m. Consequently, we were asked to obtain the best solution within this figure, the descending order of priorities being good wave conditions, current and wind. If either of the latter two had to be omitted provision was to be made for fitting them at a later date. The figure of £3m. was somewhat arbitrarily obtained, but it was felt essential to work towards a budget. In the event this financial target proved to be an enormous spur towards considering every design aspect very critically and resulted in the production of some very effective design solutions.

Integrated hydrodynamic complex

Capital cost is only one aspect of such a venture and although not part of this study, the question of an efficient and self-supporting management structure was always borne in mind. The University authorities had indicated that they would not wish to run such a commercial facility but were prepared to make over the existing experiment tank at Glasgow, plus the associated library and office space in Acre House, subject to suitable financial and other agreements about access to research facilities, etc., to a management organization which would reflect the interests of industry, national authorities and the University. Thus the complete complex would comprise:

(a) The new high wave-deep water tank, described here

(b) The existing 76m × 4.6m × 2.4m deep testing tank equipped with carriage having a maximum speed of 7 m/s, a maximum wave height of 0.3m and capable of producing regular and random seas.

(c) An office block and library housing up to 30 research staff with self-contained computer services, workshops, etc.

The experimental complex is sited on the edge of the new science park being built jointly by Glasgow and Strathclyde Universities, Strathclyde Region and the Scottish Development Agency. This area is close to major airports and served by an excellent road system. It was felt that the combination of unique experimental facilities, backed by a major university with a large marine technology research programme, would attract support from a wide range of industrial and government interests.

Site geology

Sound rock is approximately 9m below ground level and forms an appropriate foundation for the 10m deep tank. This bedrock is predominantly sandstone, but contains mudstone strata and several workable coal seams.

Consideration of the likely thickness of rock cover over the highest worked seam and of the net unloading on the ground that the new facility will impose, indicates that no grouting of these seams should be required. The grouting associated with the installation of the rock anchors beneath the tank will improve the ability of the rock to bridge over any settlement.

Factors influencing planning of the development

The main consideration has been to design the minimum capital cost facility which meets the functional requirements of the brief consistent with all relevant building and planning regulations. Within these constraints the internal layout of the facility has been optimized to ensure efficient usage and the overall location within the site chosen to provide links with both the Acre House offices and the existing hydrodynamics laboratory. This particular location has also proved beneficial from geological considerations as it enables the major part of the excavation to be in overburden, while enabling the main tank base to be founded on bedrock. It allows most of the trees on the site to be retained and provides an attractive and functional integrated complex.

Isometric and sectional drawings of the proposed facility are given in Figs. 1 to 5. The major features are the large waves and water depth available. The former are produced over 1.5m high by eight single-flap wavemakers in vertical piano-key formation along one 18m tank side and subsequently absorbed by a 25m long beach on the opposite one. The wavemakers have the ability to produce directional irregular wave spectra of general shape and have control of irregular wave periodicity up to sensibly infinite return periods. They can also produce breaking waves in a controlled manner.

The tank has overall internal dimensions of 65m long × 23m wide × 10m deep, with an additional depth of up to 10m above a movable floor in a 9.2m diameter pit. The length gives adequate distances from the test section to both wavemakers and beach. The beach is sufficiently long to absorb efficiently the longest waves generated.

Any undesirable transverse waves that accumulate are damped by the oscillating water columns along the longitudinal sides and the 23m wide tank floor beneath them permits the wide spread of moorings. A 10 tonne beam crane transfers models between the loading bay and the test area, while two transversely spanning carriages allow easy access to them from the side aprons.

Care has been taken in the overall internal planning of the project to optimize its operational efficiency. The control room is centrally placed at roof level with direct line-of-sight communication to the test area, wavemakers, crane, loading bay and optional workshop. A viewing gallery across the tank allows close inspection of experiments in progress without disturbing the control room operation. The top surface of the movable buoyant floor in the test area can be raised above water surface level to enable models to be easily and accurately attached. This floor is capable of being rigidly located at any intermediate position between main tank and pit floor levels.

Details of specialized equipment

Wavemaker

The wavemaker will be of the flap type with hinge depth approximately 5m below the still water level. There will be eight 2.2m wide sections each capable of being controlled independently to produce long crested regular waves, irregular waves and a certain amount of directional spread (18°). Wave absorption capability will be included, details depending on the particular manufacturer selected. The figures have been prepared using the wavemaker consultant's design.

There are essentially four different types of wavemaker: pneumatic, plunger, piston and flap types. Currently the flap type is more popular for making large deep water waves and it is this type that is shown here.

The most critical dimension with the flap type of wavemaker is the hinge depth. The intention is to try and match the horizontal velocity at the flap (and therefore that imposed on the fluid) with that of a deep water wave (i.e. a motion dying out exponentially with depth). Since the rate of this exponential decay depends on wavelength it can be seen that fixing the hinge depth fixes the wavelengths at which the wavemaker will perform best. The other factor regarding hinge depth is that the height of the wave will depend on the power put into the wave by the wavemaker. Obviously, to get the same power from a smaller hinge depth will require a greater rotation at the flap and hence further departure from the linear theory and poorer wave forms. On the other hand it is not possible for a deep flap to produce good short waves. As the test area is reasonably close in terms of wavelengths to the wavemaker for the larger waves, most stress has been placed on getting them right with a possible loss in performance at the shorter wavelength at the spectrum. (A double flap system with two actuators would give the advantages of both large and small hinge depths, but would be substantially more expensive.)

Fig. 6 shows some design curves for single-flap wavemakers with plots of trough-to-crest wave height against period. The left of the operating region is bounded by a line showing the steepness limit for long crested regular waves. This line intersects lines showing the limits or various power levels corrected for 10m tank depth, e.g. the design point of a 1.5m trough to crest height with a 4.38 second period can be achieved with a mean power of 10 kW/m. In a regular wave train the peak instantaneous power will be double this value.

As narrow flaps allow a greater amount of spread in the wave direction, the specification called for flaps no wider than 1m. As the design evolved, however, this was relaxed to 2.2m since the width of the tank became the governing criterion in directional spread for all but the shortest waves.

The flap material, power actuator mechanism and choice of a wet backed or dry backed design depends on the manufacturer and the merits of the various configurations were considered in some detail.

Oscillating water columns

The detailed philosophy behind the design of the oscillating water columns was established. The two major considerations are that the natural frequency of the column should lie close to those wave frequencies in the tank which it is hoped to absorb; and secondly that there is sufficient mass in the column to provide energy absorption. In the longitudinal direction it is hoped to generate waves between 4 and 30m in length. In the transverse direction the principal frequencies of interest are those connected with standing waves in the tank, i.e. wave lengths of 2b, b, 2b/3, b/2, 2b/5 where b is the tank breadth. In order to reduce the edge distortion effects when generating long crested waves it must be possible to shut them off. As there will be transverse partitions every 2m which prevent longitudinal movement of the air this will be possible by providing a means of closing the vent holes. Care will have to be taken that this will not set up a resonance with the standing waves in the tank but no problems are foreseen.

Removable walls in the test section

The downstand wall on the inner side of the oscillating water column is removable on both sides over the 12m test section. This will enable model fixing walls or wavemaker/absorber units to be inserted at a later date and greatly improve the flexibility of the working area.

Beach

Originally it was proposed to fit active wave absorbers opposite the wavemakers to absorb the waves. However, it was found that a well-designed beach would have lower reflections and cost less. The most important parameter in the design of a beach is its length. A 25m long beach has been provided giving a total tank length of 65m. The bottom of the beach will slope from a depth of 10m in the tank to water level at the other end and it will be fitted with wave absorbing material, and possibly even a sophisticated oscillating water column arrangement. It is anticipated the wave reflections from the beach should be as low as 2%.

Carriages

Two carriages will be provided to enable access to both sides of the model. They will be powered by electric motors and be capable of being locked in position. One of the carriages will have a platform which can extend right down to water surface for ease of access to the model. Both carriages will be adequately equipped with power points, etc.

Crane

The 10 tonne gantry crane will cover the test section and the workshop/assembly area. It will be capable of being controlled from either the apron or from one of the carriages.

Movable floor

The movable floor will have positive buoyancy and will be lowered into position by means of wires which pass through blocks on the bottom of the pit and are led to electric motors on the surface. The lower part of the floor is conical in section to aid with location in the pit when being lowered into position. The clearance between the floor sides and the pit wall is 0.1m.

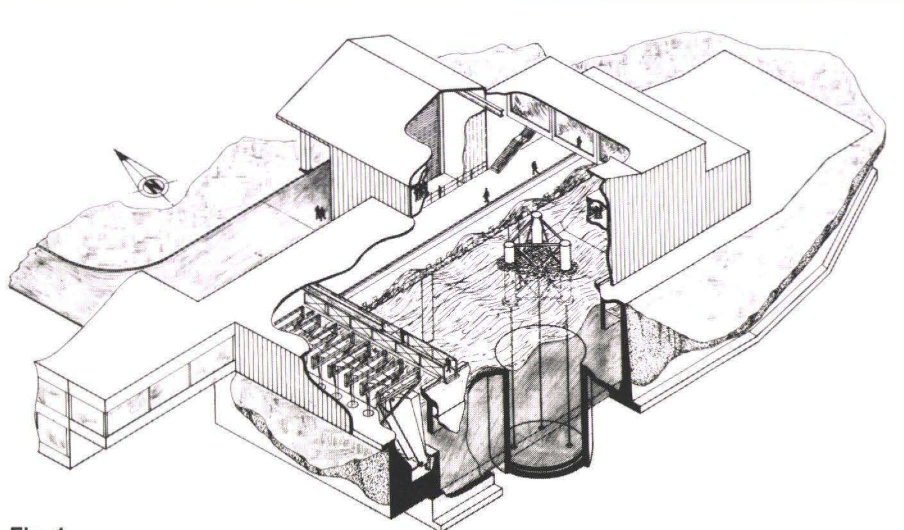


Fig. 1
Isometric view

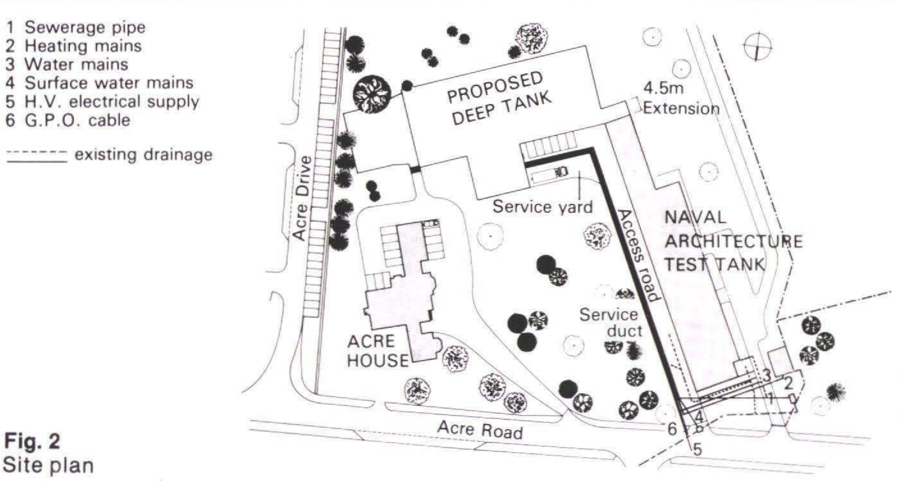


Fig. 2
Site plan

The floor will be used for testing models such as jacket structures, tension leg platforms, guyed towers, etc., which have to be attached rigidly to the bottom. It will be capable of being floated to the surface for installation of a model when required. Due to the limitation in head room some structures will have to be added in sections. In these cases the floor will be lowered till the top of the section is just above the water line to enable the subsequent section to be added.

Tests will be able to be carried out with the floor fixed rigidly at any depth in the pit. Four 1m diameter access hatches have been provided to enable divers to pass through the floor to work in the pit area if required.

Computer
The computer system based on a DEC 11/780 will be able to analyze data in real time from both tanks and carry out normal time-sharing operations (for program development, etc.) simultaneously.

Possible future options

Current
Should the need for a transverse current arise at any time after the completion of the tank it can easily be satisfied. Ducts would be fitted below the oscillating water columns which would carry the current around the perimeter of the tank and this would be diffused over the desired area through pipes extending nearly to the surface. The effectiveness of the oscillating water columns would not be reduced.

Current in the pit would be provided by pumping water through a double bottom in its base. This would diffuse into the working area through tubes.

Wind
Wind could be provided by a batch of fans which would be mounted on one of the carriages. Provision for their mounting has been made in the carriage design.

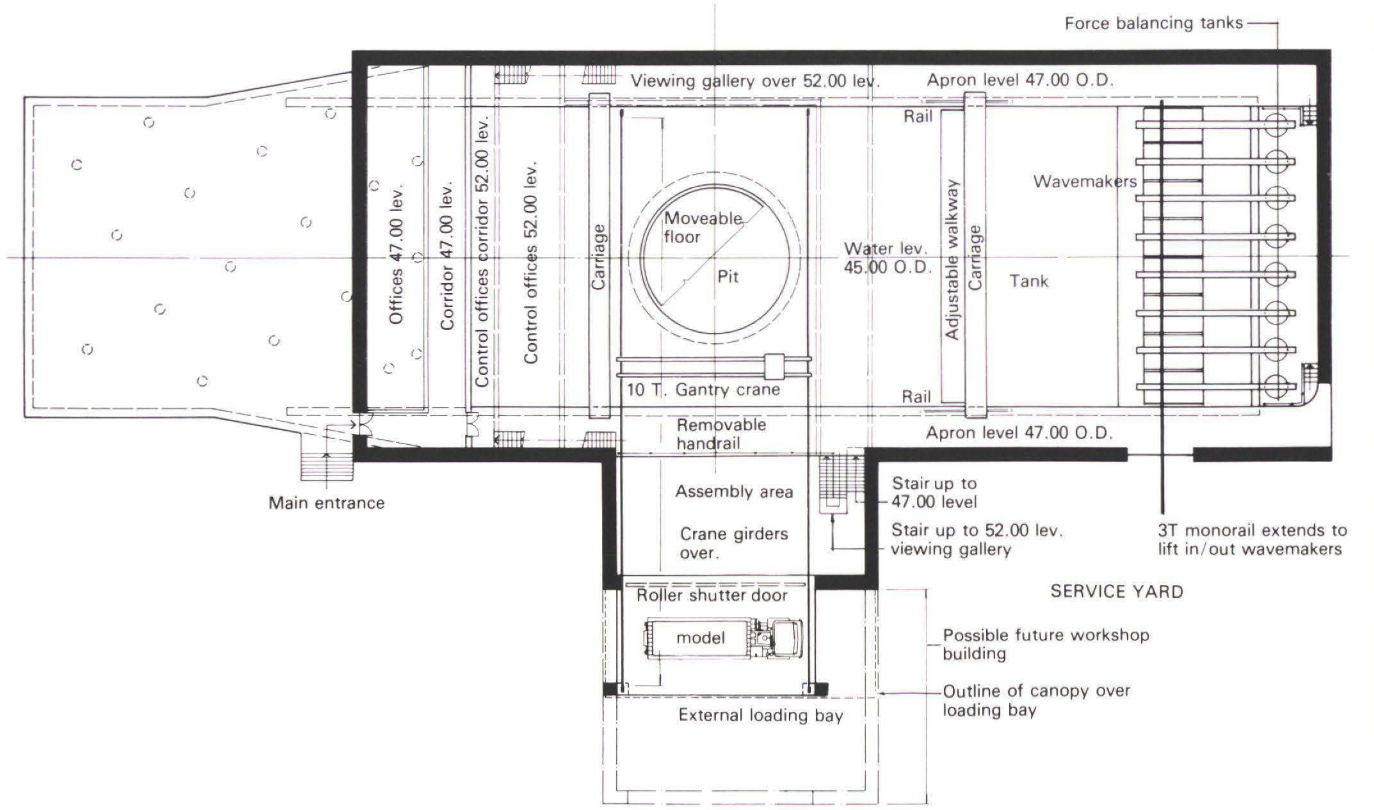


Fig. 3
Plan of proposed facility

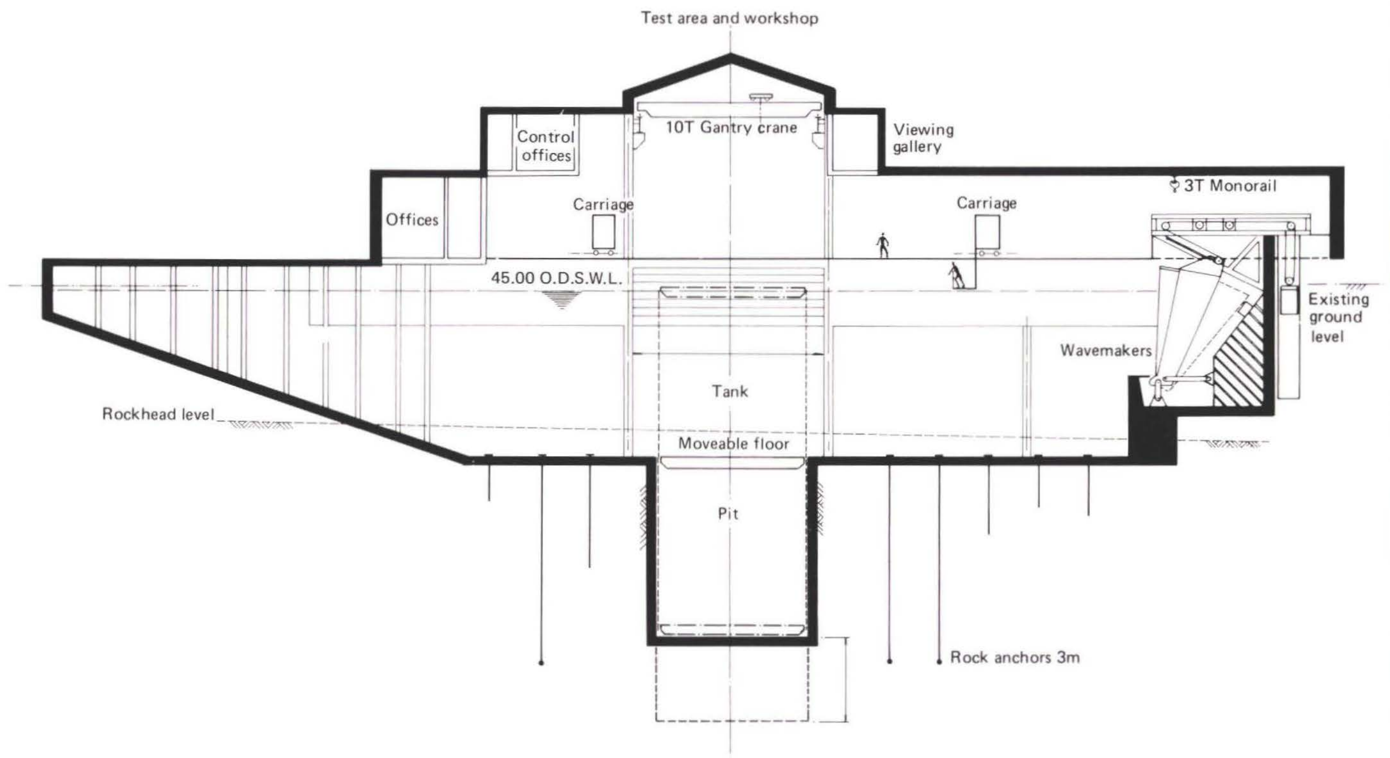


Fig. 4
Longitudinal section

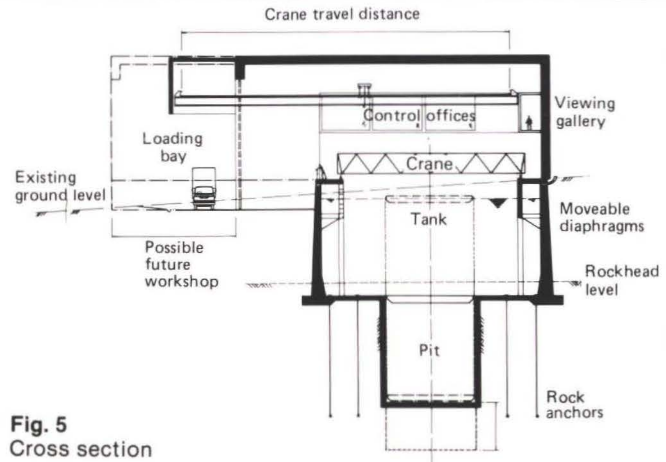


Fig. 5
Cross section

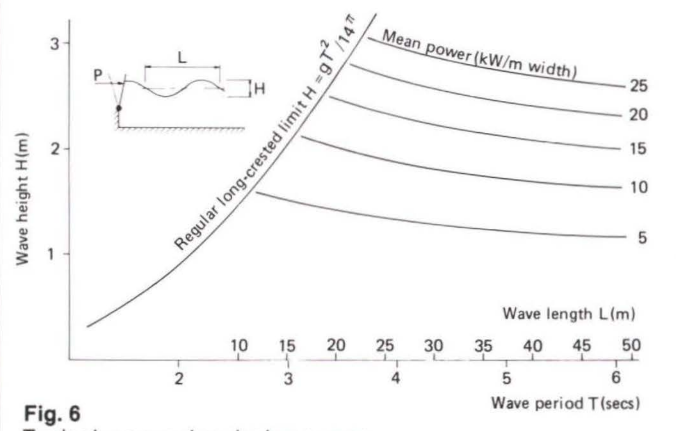


Fig. 6
Typical wavemaker design curves

Workshop

Should a workshop prove necessary at any time after the completion of the building this is possible by simply extending the assembly area to a total of 23.5m from the side of the tank building. The gantry crane would be extended and the loading bay incorporated within the building extension as shown in Fig. 5.

Conclusion

The high-wave, deep water tank will have immediate benefits for the offshore oil and gas industry and for that reason a major share of the capital cost is being sought in that direction.

The proposed commercial structure of management and operation will be a new departure for offshore industry/university co-operation, but is one which accords well with the change in public policy over recent years.

While its commercial viability must relate to the technical and financial problems of today, an experimental facility of the type being discussed would have a life far beyond the problems that engineers are currently contemplating. The availability of mineral and other raw materials on land is shrinking, due both to consumption and to political restraints, and the oceans of the world offer the maximum scope for expansion in the next century. If the UK is to have a major part in such activities it must invest in the basic experimental facilities which will provide the essential scientific data for the new technologies required. The proposed tank represents one stage along that road and it is hoped that industry and government will look ahead past the present time of recession and invest for the future.

Acknowledgements

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