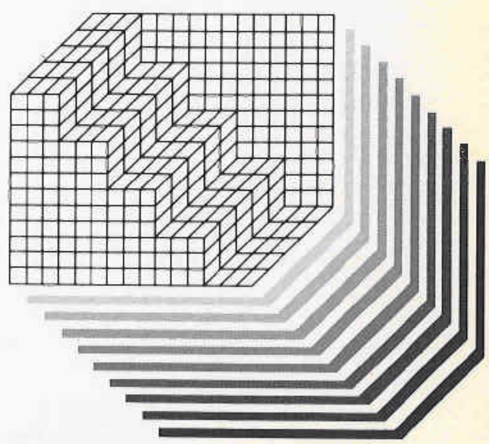
Patterns 3



Patterns 3

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John Morrison C Eng, FICE, FIStructE

John Morrison, a founding partner in Buro Happold, has recently celebrated his 50th birthday. During his first 25 years of engineering, he has undertaken some very fine and challenging engineering projects. This article reviews some of his achievements to date.

On leaving school in 1954 John joined Bracknell New Town Development Corporation as an articled pupil to the Chief Engineer, Mr J T Kendal, OBE, CEng, FICE, at an annual salary of £180. In the years 1955–1959 he attended day release education achieving honours in all subjects whilst gaining engineering experience in land surveying, road construction and sewerage. The conclusion of his pupillage coincided with the launch of the Russian sputnik and the recognition by the U.K. Government that Britain was in danger of falling behind in engineering technology. Scholarships were being offered for full time education, and John was accepted for the Diploma in Civil and Structural Engineering course at Brighton Technical College.

Upon graduation in 1961, he joined Ove Arup and Partners who at that time were about 200 strong. There, John was a member of the team working with architects Sir Basil Spence, Bonnington, Collins on the Swiss Cottage Swimming complex in London. The project comprised two olympic size pools, a training pool and gymnasium and was itself a significant and interesting project, but the design of the roof provided perhaps the most challenging aspect. Spans of 30–40m were constructed using post tensioned concrete beams, which at the time involved some quite advanced engineering practice.

On completion of that project in 1963, John became the sixth member of a group in Ove Arup and Partners headed by the then senior engineer Ted Happold. That move was the start of a most fruitful collaboration that has continued to the present day in Buro Happold. It commenced with work on a number of Exeter University projects in reinforced and precast concrete, again with Sir Basil Spence, Bonnington, Collins. This was followed by a wide range of projects including the interesting timber and tie rod roof for Bootham School Hall with architect Trevor Dannatt (Ref 1.1) (Fig 1.1).

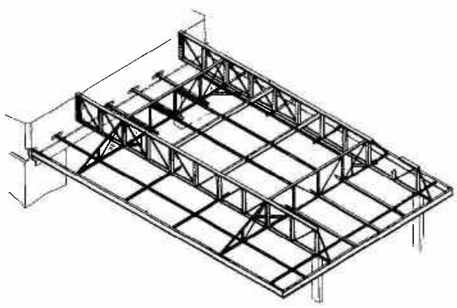


Fig 1.1 Roof structure, Bootham School

The late 60's saw a significant change in both John's role within Ove Arup and Partners, and the nature of projects undertaken by him there. In Ted Happold's group, he became the structural engineer responsible for all projects commissioned by Lambeth Borough Council. Under the direction of the Borough Chief Architect, Ted Hollamby, the pace of social construction at Lambeth was hectic with numerous projects running consecutively, and exceeding tens of millions of pounds in total, at today's prices. The task of rehabilitation was wide ranging and included housing estates, high rise council flats, old people's homes, day care centres, health centres, meal distribution centres, schools and industrial units.

Working with the architects of Lambeth, Ted Hollamby, Don Eastaugh, Bill Jacoby, Charles Attwood and George Finch, John's projects used a wide range of engineering materials, and included Walnut Tree Walk, West Norwood Library, Somerleyton Road Training Units, West Norwood Civil Defence Centre, Lambeth Road Flats and Central Hill Stability and Housing (Ref 1.2, 1.3) (Fig 1.2). Indeed I can remember well at least twenty such consecutive projects at this phase of John's career since, in 1968, I inherited them in their various stages upon my return from Africa.

Under advice of management consultants Kinseys, Ove Arup and Partners then divisionalised and Ted Happold became Executive Partner of Structures Division 3 supported by six senior engineers. John was one of these six (Ian Liddell and I, now his partners, were amongst the others). John moved to take charge of the first scheme for the redevelopment of the London Docklands. This Dockland project, comprising the St Katharine Docks and the Tower Hotel next to Tower Bridge, was with architects Renton Howard Wood and developers Taylor Woodrow (Ref 1.4). John was principally involved with the 830 bedroom hotel, which qualified for a government grant of £1,000/bedroom, providing that completion was achieved by mid 1970 (Fig 1.3). Foundations were large diameter, under reamed piles founded on the London Clay. The superstructure bedroom walls were designed as deep beam diaphragms between the outer columns. The impressive mezzanine floor was achieved by taking support from a series of deep beams and hanging tension walls, thereby creating the column free space at ground level beneficial to the operations of the hotel's public areas (Fig 1.4).

As Tower Hotel neared completion, Ted Happold and Peter Rice, both of Structures 3 of Ove Arup and Partners, together with architects Piano and Rogers, presented the winning entry for the international competition for Centre Pompidou in Paris (Ref 1.5) (Figs 1.5, 1.6). To maintain the impetus from the competition, it was decided under Ted's guidance to start construction as soon as possible. A very big hole in the ground, needed for the underground car parking, established site possession. This strategy required a rapid start to construction before the design of the famous superstructure was fully developed. John was invited to move to Paris to assume leadership of the Arup's team responsible for the substructure design and site modelling. The architects split their

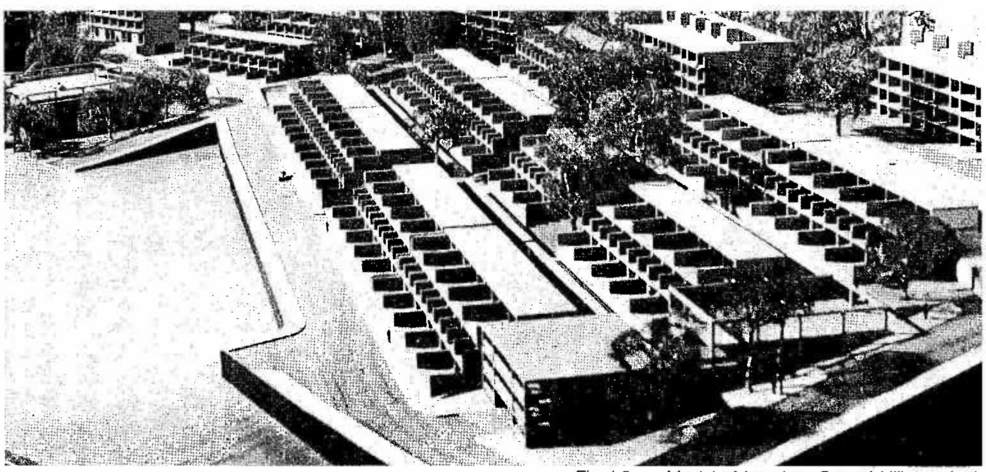


Fig 1.2 Model of housing, Central Hill, Lambeth

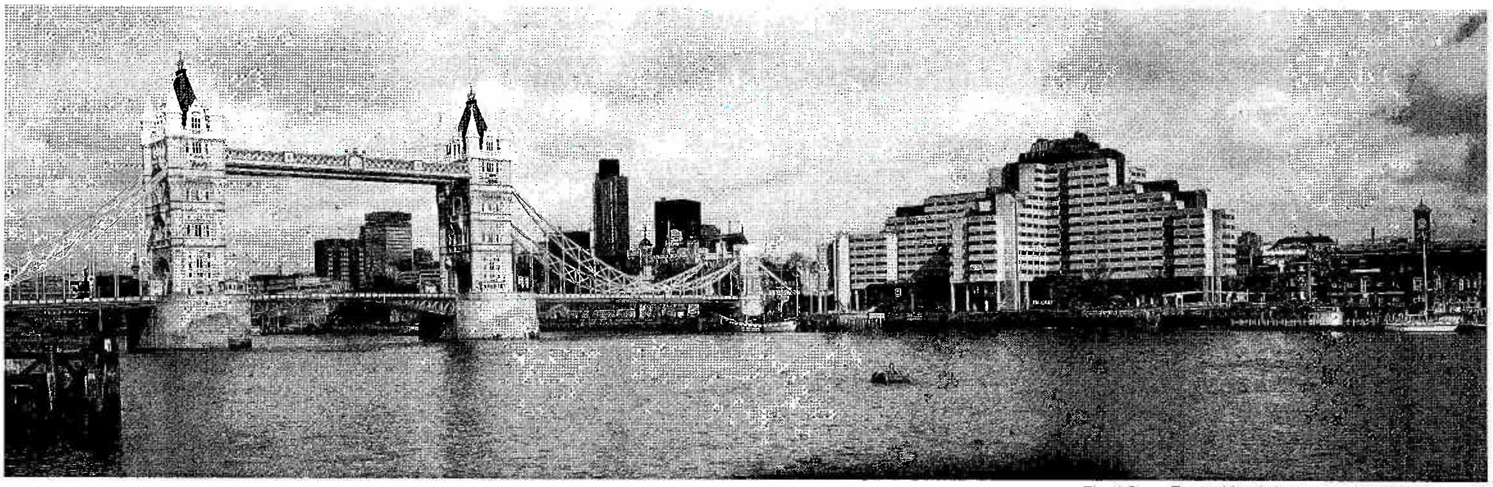
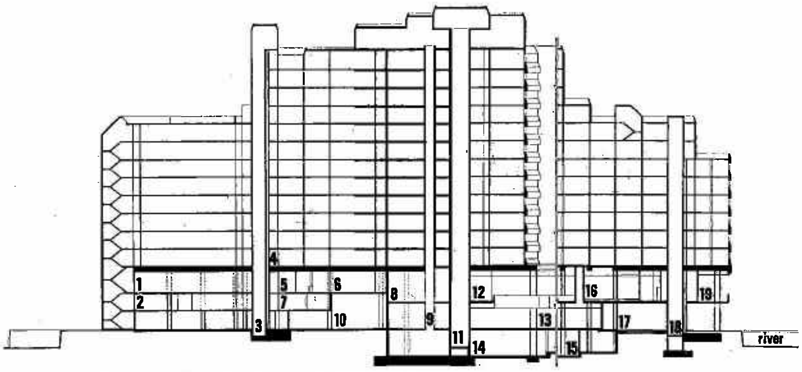


Fig 1.3 Tower Hotel, London, viewed from south



broken north-south section (scale: 1/64in = 1ft)

1. office corridor
2. staff corridor
3. lift shaft
4. staircase lobbies
5. offices
6. staff dining
7. female staff
8. soft furniture store
9. services riser duct
10. north road
11. passenger lift shaft
12. gallery
13. two-storey entrance foyer
14. boiler room
15. plant
16. hotel bar
17. south road
18. firemen's lift shaft
19. terrace

Fig 1.4 North-South section through Tower Hotel

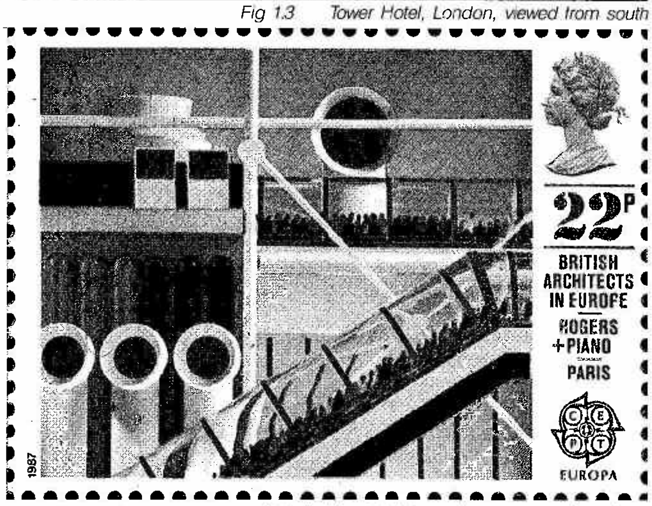


Fig 1.6 Centre Pompidou, "British Architects in Europe" commemorative stamp

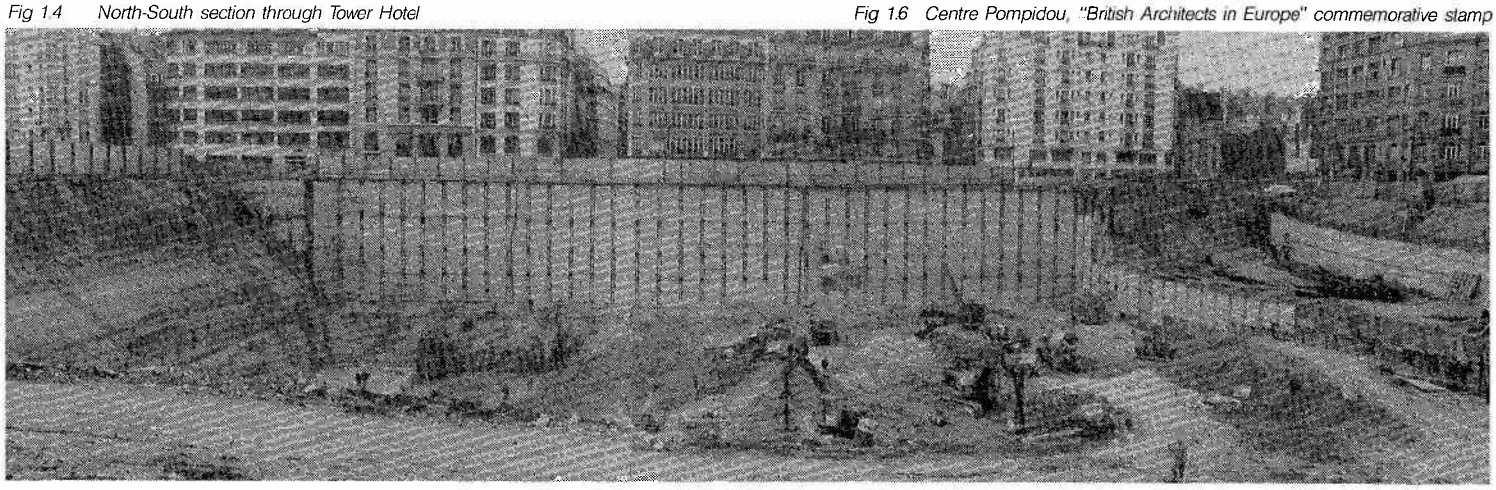


Fig 1.5 Site excavation for Centre Pompidou, Paris

team similarly and a series of subcontracts were programmed to achieve the task. Within two months, and often working through the night, a contract for the first 6m of excavation was let. This was followed by a grout curtain contract for a 4m wide strip around the 180m × 100m site, which in turn was followed by the concluding excavation contract for the residual 14m. With a final depth of some 20m below the Metro Line in Rue de Renard, considerable analytical problems were encountered in the design of temporary walling and the necessary tie backs.

Construction proper commenced with the installation of vast diaphragm wall panels as barrette foundations designed to take both the vertical load and bending moments induced by the proposed superstructure. It was imperative to mobilise the skin friction on the installed barrette, a design feature never before achieved in France. Before leaving Paris in 1972, the substructure team, under John's direction, had completed the design and let the contract for the construction of the interior foundations and internal structure to podium deck level. This was to house cinemas, lecture halls, roads and parking for 800 cars, and a piazza strong enough to support a railway engine. It was a good base upon which to found the better known superstructure. (Ref 1.6).

On his return to London John undertook a varied range of projects including a series of designs for the technological development of the Post Office. With architects Scott Brownrigg and Turner, a programme of mechanised letter sorting offices and parcel concentration offices was realised, and the design for Portsmouth Head Post Office was undertaken. During this time John visited Kuwait, following a request from the Kuwaiti Ambassador to assist with the investigation of the collapse of a large multi-storey complex. The preliminary investigation concluded that the problem could be attributed to the grade of concrete supplied. It was also clear that the further two phases, which were well into construction, were themselves at the point of failure. Construction was halted and emergency supporting temporary works were designed and installed. Remedial designs were then prepared and constructed over the next two years under a supervisory team of eight despatched to Kuwait to monitor the exercise.

In 1975, following a telephone call between Ted Happold and Dr Kamel Kafrawi, (the Egyptian architect who had also submitted an entry for Centre Pompidou) John, as a newly appointed associate in Ove Arup and Partners, was put in charge of the structural design team for the 6000 student Gulf University in Qatar (Figs 1.7, 1.8) (Ref 1.7). As Chief Design Engineer he was responsible for the coordination of design matters with the collaborating architects, Renton Howard Wood

Fig 1.7 Qater University, commemorative stamp, February 1976

Levin and services consultant, DSSR. The task was enormous and the design concept imaginative. The lack of construction technology and skilled manpower in Qatar led to a precast concrete solution which itself required the design and construction of a major precasting factory before construction of the University could commence. Indeed, it proved to be the birth of a national industry in Qatar since on completion of the University the precasting facilities were made available to the country at large.

In the intervening period up to 1976/7, John was fully committed to the realisation of the University and at the peak some 240 staff were employed on the design. Indeed John's involvement was so critical that he stayed with Ove Arup and Partners to see the contract documentation completed and

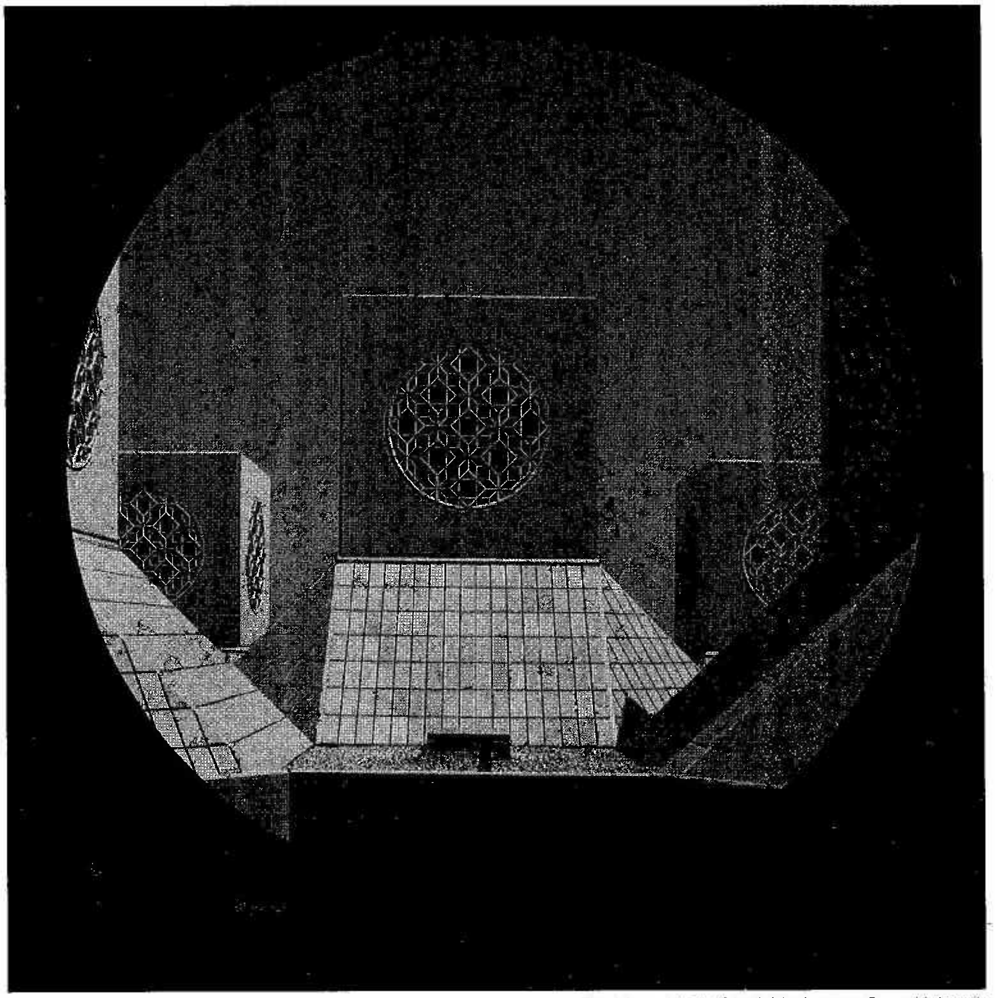


Fig 1.8 View of individual tower, Qater University

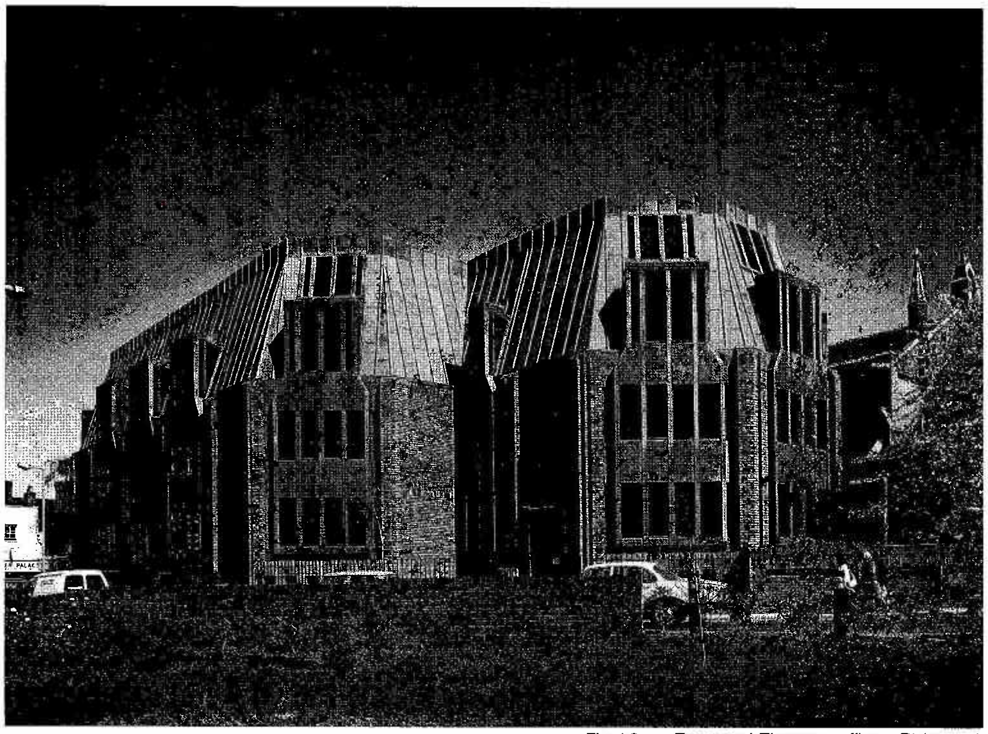


Fig 1.9 Townsend Thoresen offices, Richmond

the first phase of the University construction under way before leaving to devote his full energies to the foundation of Buro Happold.

In Bath, meanwhile, the Buro Happold practice was growing and, whilst commissions were not as large as those to which John had become accustomed, a great variety of projects were being undertaken. For his share, John undertook the Richmond office development for Townsend Thoresen Properties (Fig 1.9) and the refurbishment of the adjacent site of the church of St John the Divine. Both projects were with architects Dry Butlin Bicknell. In Bath, he also had opportunities to renew his relationship with Kuwait and in particular with a very special client of our practice, Mr Khalid Al Marzook. He was involved with the design of the Al Marzook family villa, a substantial complex in reinforced concrete — with an interior in the traditional Arabic style. This was followed by a project for the same client — the Islamic Medical Centre, with its astonishing level of precast concrete finish and major dome as illustrated in Patterns 1 (Ref 1.8). Both projects were with the architect Isaac Zekaria. The Islamic Medical Centre is currently a candidate for the Aga Khan Award.

Within our practice, John is respected for his skill in organisation and management. In all probability, this dates from the earlier period of his career when he had the opportunity to develop these skills while running the Lambeth projects. However, John has also always been interested in new concepts in structural design and in engineering efficiency. He has been particularly active in the development of methods of composite construction and accepted an invitation to assist the BSI in the development of a new Code of Practice for Composite Design. His work on that document has subsequently led to his chairmanship of the British committee for Eurocode 4: Composite Design.

Early in 1981, Buro Happold was invited to submit an entry for a structural solution for the design of the Tsim Sha Tsui Cultural Centre in Hong Kong, (Fig 1.10) with predominant emphasis on the unique roof that had been proposed by the Architectural Department of Hong Kong. We were one of three firms reaching final selection in Hong Kong. With Terry Ealey developing the design of the hanging roof, and Ted Happold's contribution, Buro Happold was successful in being appointed as structural engineers for the project. This required the setting up of an office in Hong Kong and with Hong Kong engineer Tom Ho, the office of Ho-Happold was established. John and Terry Ealey transferred to that office in 1981 and for three months jointly developed the design, with Terry concentrating on the cable net roof and John on the remainder of the building comprising the Lyric Theatre, concert hall and studio theatre. In all the complex seats a total audience of 4,500. The target cost of the project was £38 million.

Terry then returned to Bath to undertake the detailed analysis and design of the roof, while John remained in Hong Kong to continue the coordination of the total process. Whilst the roof design was challenging, the superstructure and foundation designs were equally taxing and John had to draw on all of his experience gained in preceding years. The site was largely a reclaimed coastal strip and foundations were necessarily mixed, including barrette walling, Franki piling and pads directly on to rock. The foundation contract alone had a value of £2.6 million and was let on programme (Ref 1.9). Following construction of a substantial portion of the foundations, architectural replanning was required which in turn necessitated complex recalculation of the existing foundations in an attempt to underwrite the change of design and the resulting incompatibilities. It is a tribute to John's organisational skills and a direct consequence of the design effort expended in overcoming such a major problem, that no claim was received from the contractor for the foundations on the basis of late information.

The superstructure design called for the creation of dramatic volumes with virtually no visual obstructions within the auditorium. Upper seating balconies were therefore cantilevered and foyer areas formed by intersecting deep beam walls. The project is due to be opened later this year and will no doubt be reviewed in detail in both the architectural and technical press.

In parallel with the design of the Cultural Centre, John also undertook commissions for several projects for the extension of the Hong Kong Metro System. Causeway Bay Station was one of these projects, where excavation was taken down to 80m through decomposed rock and solid granite. The construction method was "top down" and the temporary works design was included within the commission. Other projects included the competition winning design of Tuen Mun Technical College with architect Kenneth Kan. This £8 million complex was both designed and constructed during John's four years residency in Hong Kong.

Since returning to Bath in 1986 John has rebuilt his project team and renewed old acquaintances resulting in new commissions both in structural and building services design. Recent projects include complexes for Schwarzkopf Ltd and Instron Ltd with architects Denton Scott Associates; the refurbishment of the Royal York Hotel, Bath, and the restoration of the 17th century Angel Hotel in Chippenham with architects Dry Butlin Bicknell.

Projects currently in progress include the

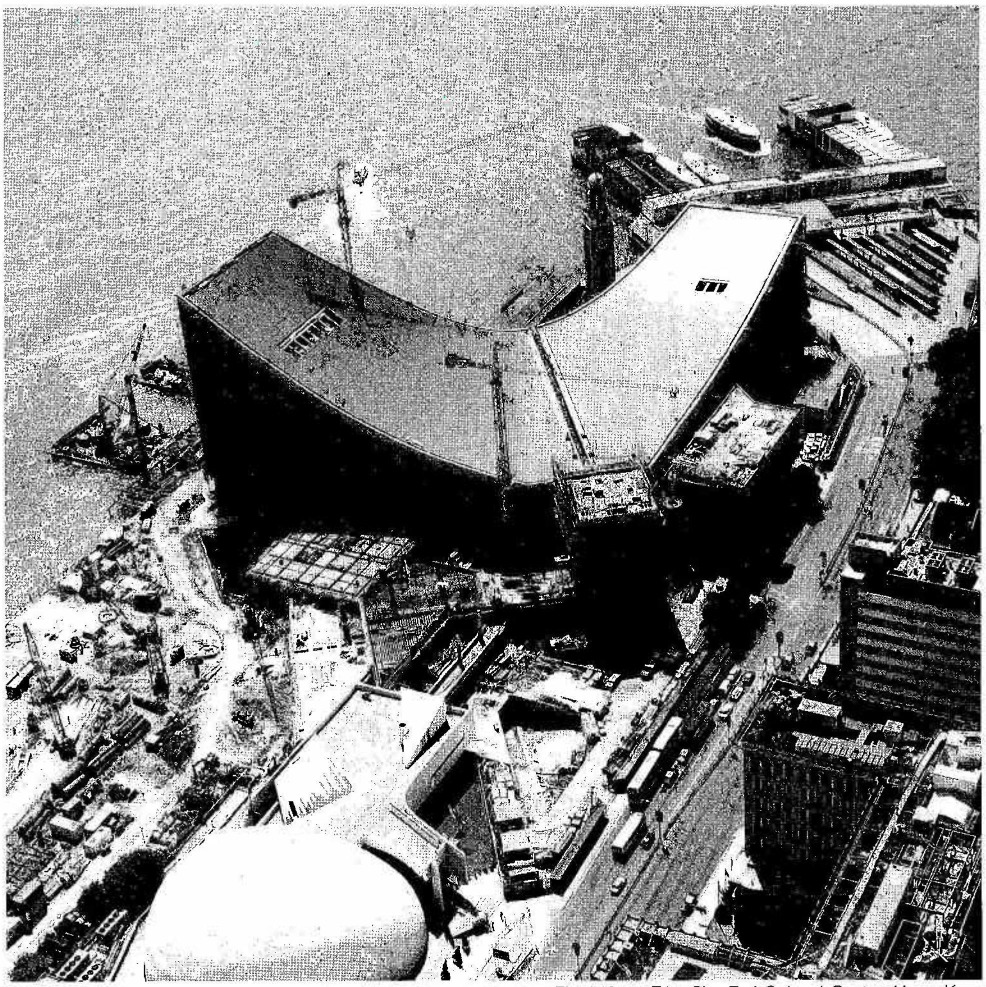


Fig 1.10 Tsim Sha Tsui Cultural Centre, Hong Kong

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redevelopment of the flagship of the Odeon Group at Kensington — a complex of five underground cinemas and parking, with multi-storey offices and apartments above, with Bowerbank Brett and Lacy as the architects; a new office development in Luton; and recently, confirmation of a commission for the refurbishment of a number of tired blocks of council flats built in London in the 60's. Together with special responsibility within the practice for engineering computer software (Ref 1.10), a variety of the practice's standard specifications and external committee work, John has a busy life, leaving little time for other activities, although he is always good for a tip on the stockmarket!

A review of John's professional career would not be complete without mention of his wife Wendy and their children David, Clive, and Karen, now grown up and following their own chosen careers. Engineers necessarily move to the project site and an international engineer such as John spends much time away from home demanding a tolerant and understanding family. John has been well supported in this need.

We all wish you, John, a most happy half century year, salute your achievements to date and look forward with anticipation to your work in the years ahead.

To be continued . . .

Peter Buckthorp

The Design of Ground Slabs for Industrial Buildings

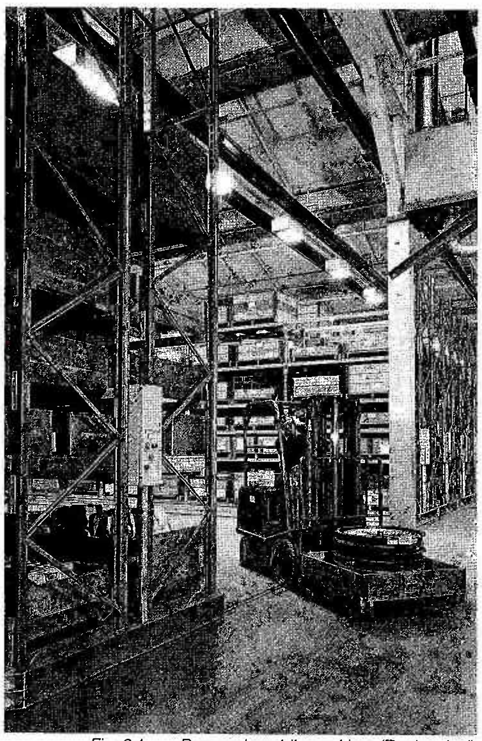

Fig 2.1 Powered mobile racking (Dexion Ltd)

The Problem

Ground slabs have, until quite recently, usually been designed by empirical methods. This is acceptable for preliminary sizing, but as warehouses become larger and floor loading increases it is clear that the old methods are no longer appropriate and rigorous new methods of design are required. Storage systems are now various including bulk storage, racking, mezzanine storage, automated picking and packing, through to mobile racking (Fig 2.1). Furthermore, the high reach fork lift trucks and mobile racking commonly in use have exacting operational tolerances. Combined with these requirements it is often found that warehouse sites will be on derelict land or in locations with poor subgrades.

Floor Loadings

Discussion in the Journal of the Institution of Structural Engineers during the early 1980's drew attention to the fact that the current CP3 Chapter V Part 1 did not adequately cover existing operational conditions. The substitution in 1984 of BS 6399 Part 1 also failed to properly address the warehouse loading requirements so much so that in 1987 the BRE brought out a review document (Ref 2.1) in which it was stated that BS 6399 loads should be regarded as minimum values.

Some supermarket chains now specify that a floor loading of 36 kN/m be adopted as blanket load for their retail outlets. For bulk storage BRE recommend that the values given in Table 2.1 should be adopted, adding a warning that for general warehousing the nature and method of loading could vary significantly throughout the life of the building.

In general the design of warehouse floors using a uniformally distributed load is not realistic. The significant design factors now are the distribution and magnitude of the individual point loads arising from the use of racked storage. This change to racked storage is a comparatively recent phenomenon, although sufficient time has elapsed for the failures of the traditionally designed ground slabs supporting such racking to start manifesting themselves. Common problems are excessive cracking and/or deflection due to both the underestimation of the operational point loads from the racking systems, and the use of low strength concrete which has low resistance to cracking. Table 2.2 gives the present BRE recommendations for static and mobile racking and warns that the upper values are by no means uncommon.

Most damage occurs if the racking is located on the corner or along the edge of slab joints. For this reason the position of construction joints should be carefully specified and dimensioned on the layout drawings. These drawings should be issued to the racking manufacturer with a request that his layout take note of these details. Preferably this racking layout should be agreed before the design of the floor slab.

Tolerances and Sub-Base Construction

There are two main types of failure, slab cracking and deflection. For some operations, such as automated warehouses with high bay picking and packing, or with mobile racking, deflection is the most important criterion. Equipment manufacturers call for floor or runway rail gradient limits varying between 1/1000 and 1/3000. There are few subgrades that can ensure this accuracy without recourse to additional methods of support, such as stone columns or piled runway beams. For racked and bulk storage systems the principal consideration is the safe operation of the fork lift trucks and here it is normal for operators to request gradient limits between 1/500 to 1/1000 (Ref 2.2). It should be noted that these limits are more exacting than those normally adopted on the suspended floor slab of an office block.

On clay soils settlement calculations should always be carried out especially if the design floor load exceeds 50% of the allowable bearing capacity of

Table 2.1 **Weights of Stacked Palletised Goods (BRE)**

	Wt/Unit Volume (kg/m^3)	Uniformly Distributed Load (kN/m^2) (4 Pallets 3* Sacks High)	Uniformly Distributed Load (kN/m^2) per metre Height
General goods including porridge oats, pet food, washing machines, rolls of newsprint, etc	200 to 800	8 to 40	2 to 7.5
Fertiliser or raw materials in large polypropylene sacks, 500 or 1000 kg each	1000	30	10.0

*Note that there is risk of instability if unpalleted sacks are piled more than 2 or 3 sacks high, but it is not unknown for them to be stacked 5 high

Table 2.2 **Load Capacity of Standard Racking (BRE)**

	Uniformly Distributed Load (kN/m^2) per Metre Height		Total Concentrated Load on Pair of Adjacent Rack Leg Base Plates (kN)	
	Typical	Upper Value	Typical	Upper Value
Static racking	4.4 to 6.7	36.0	90 to 140	225
Mobile racking	4.4 to 6.7	36.0	60 to 120	360
Static shelving	4.0 to 8.0	40.0	5 to 10	50
Mobile shelving	4.0 to 8.0	40.0	–	44

the subgrade. When a soil is stressed to its allowable bearing capacity there is likely to be a resultant settlement of 20–25mm. This will be amplified if the surrounding foundations have a lower ratio of applied load to allowable bearing stress.

At the very least trial pits should be dug and tests carried out so that the soil type and properties can be identified and to determine the cohesion value, c, and liquid limit of the soil. The allowable bearing capacity, with a factor of safety of 3, will be:

$$q_a = 5c/3$$

The stress intensity on the soil due to point loads can be reduced by using a well compacted type 2 aggregate sub-base. Crushed brick or other coarse material should not be permitted as the points of contact with crush under load will be forced into the clay surface resulting in excessive settlement. On poor subgrades it may be necessary to adopt either subsoil stabilisation or the use of a lean concrete sub base.

Design Method

In the past, several methods of design have been proposed (Ref 2.3) which usually rely on a knowledge of the modulus of subgrade reaction. This modulus is difficult to measure but fortunately a 100% error in its selection only results in a 10–15% error in the calculated bending moments. However, the correct value must be used for the assessment of deflection. Nevertheless in certain circumstances, and provided that its limitations are appreciated, the modulus of subgrade reaction method can still be of use. A suitable modulus value, K, can be determined from:

$$K = 120 \times q_a$$

Alternatively the values given in Table 2.3 can be adopted. These should then be used in conjunction with grillage software to determine the slab bending moments.

A more direct method avoiding the need for computer analysis has been developed by Chandler of the Cement and Concrete Association (Ref 2.4). However, this method does not control settlement but is merely a way of determining slab thickness and of controlling cracking under the influence of point or uniform loads for various configurations of the slab joints. The method is based on finding a slab depth which is sufficient to ensure that the flexural stress of plain concrete is not exceeded. Mesh reinforcement plays no part in the analysis and should be determined on the basis of a further C & CA report by Deacon (Ref 2.5). This report outlines the control of shrinkage cracking in relation to the size and slab thickness as indicated in Table 2.4.

Table 2.3 **Assumed Modulus of Subgrade Reaction (K) for Typical British Soils (BCA)**

	Liquid Limit	Typical Soil Description	Subgrade Classification	Assumed K (MN/m^3)
Coarse-grained soils		Gravels, sands, clayey or silty gravels/sands	Good	54
Fine-grained soils	<50	Gravelly or sandy silts/clays	Poor	27
	>50	clays, silts	Very poor	13
		Organic soils (peat, etc.)		

Table 2.4 **Free Movement Joint Spacing and Fabric Reinforcement for Slabs (C & CA)**

Fabric Reinforcement According to BS 4483	Longitudinal Direction of Slab — Maximum Spacing of Free Movement Joints (m) — Slab thickness (mm)					Transverse Direction of Slab — Maximum Spacing of Free Movement Joints (m) — Slab thickness (mm)				
	125	150	175	200	225	125	150	175	200	225
Unreinforced	–	6 18*	6 18*	6 18*	6 18*	–	6 18†	6 18†	6 18†	6 18†
A142	25	21	18	16	14	25	21	18	16	14
A193 B196	34	28	25	21	19	34	28	25	21	19
A252‡	44	37	31	28	25	44	37	31	28	25
B283	49	42	36	31	28	34	28	25	21	19
C283						18†	18†	18†	18†	18†
B385	67	56	48	43	37	34	28	25	21	19
C385						18†	18†	18†	18†	18†
A393‡	69	58	49	44	38	69	58	49	44	38
B503‡	–	74	64	55	49	–	37	31	28	25
C503						18†	18†	18†	18†	18†
C636	–	–	81	74	65	–	–	18†	18†	18†

* Induced joints at 6m maximum centres.

† 6m maximum strip width.

‡ With these fabrics, high-yield tie-bars of equivalent cross-section to the fabric should be used in longitudinal joints when maximum spacing of movement joints is adopted in the transverse direction.

EXAMPLE:
A slab 175mm thick is 60m long between free isolation joints abutting the walls. It is to be laid in a continuous strip with induced joints at 10m centres.
From above, long mesh C503 is required. If a debonded dowelled contraction joint is introduced at mid-length, movement joint spacing reduces to 30m, and the reinforcement could be reduced to square-mesh A252, or long-mesh C283.

The reinforcement should be placed not deeper than 50mm from the top of the slab to control the distribution of cracking on the slab surface. Normally it is not considered practical to provide top and bottom mesh reinforcement. However, in extreme conditions longitudinal and transverse tensioning of the slabs using unbonded cables may be a useful method of increasing the flexural strength of the slab.

The flexural strength of plain concrete is a function of aggregate type and size. It is unlikely that the floor slab will be loaded before 90 days and therefore 110% of this 28 day value is adopted for the design. Table 2.5 gives a range of typical 90 day values.

Table 2.5

Cube Strength @ 28 days N/mm^2	**Flexural Strength @ 90 days N/mm^2**
25	3.6
30	4.1
35	4.7
40	5.3

The design should be carried out using ultimate load methods with a load factor of 1.5 on static loads and 2.0 on dynamic loads, such as fork lift trucks. As with the previous method the modulus of subgrade reaction, K, should be determined from Table 2.3. If the water table is less than 600mm below the formation level then the value of K should be reduced by 40%. The use of a sub base can enhance the basic K value of the subgrade and Table 2.6 from a report by Chandler and Neal (Ref 2.6) gives suitable values. Chandler and Neal also demonstrate in Table 2.7 how the thickness of the ground slab can be modified if a C20 lean concrete sub base is adopted.

Uniform Loading

Figure 2.2 illustrates the effect of uniform loading either side of an aisle, and the resulting longitudinal crack due to hogging of the aisle slab. For a given slab thickness, modulus of subgrade reaction and concrete grade, the radius of relative stiffness can be determined from Table 2.8. Using this radius of relative stiffness the critical bending moment for a loading of 1 kN/m^2 can be determined from Figure 2.3. The bending moment found should then be multiplied by the uniform load and a 1.5 load factor to give the design moment, M.

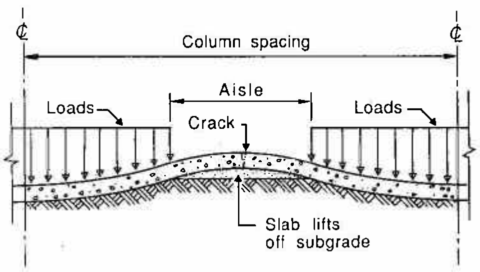

Fig 2.2 Section through aisle showing typical crack (ACI)

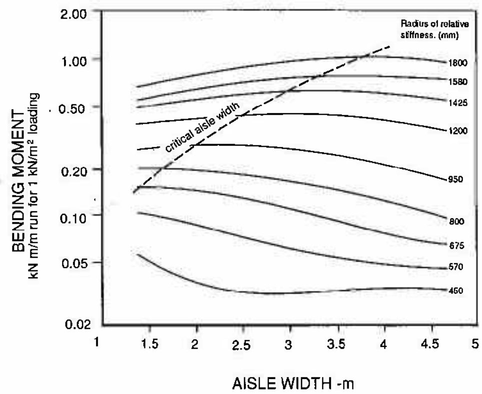

Fig 2.3 Critical bending moment for a loading of 1 kN/m2 (C & CA)

Table 2.6 **Enhanced Value of K when a Sub Base is used (BCA)**

K value of Subgrade Alone (MN/m^3)	Enhanced value of K when used in conjunction with Granular Sub-base of Thickness (mm) 150	200	250	300	Cement-bound Sub-base of Thickness (mm) 100	150	200	250
13	18	22	26	30	35	50	70	90
20	26	30	34	38	60	80	105	–
27	34	38	44	49	75	110	–	–
40	49	55	61	66	100	–	–	–
54	61	66	73	82	–	–	–	–
60	66	72	81	90	–	–	–	–

Table 2.7 **Modified Thickness of Concrete Slab with a C20 Lean Concrete Sub-Base (BCA)**

Calculated Thickness of Slab (mm)	Modified Thickness of Slab (mm) when used in Conjunction with Lean Concrete Sub-Base of Thickness (mm) 100	130	150
250	190	180	–
275	215	200	–
300	235	225	210

The required flexural strength is given by:

$$\frac{6M}{h^2}$$

(where h = overall depth of slab)

Hence by reference to Table 2.5 the required grade of concrete can be found. Alternatively the grade of concrete can be held constant and the calculation repeated to find the depth of slab.

Point Loads

In calculating point load, the initial step is to determine the equivalent radius of the baseplate area, r:

r = contact area

If back to back racking is adopted the area of the two base plates should be added before deter-

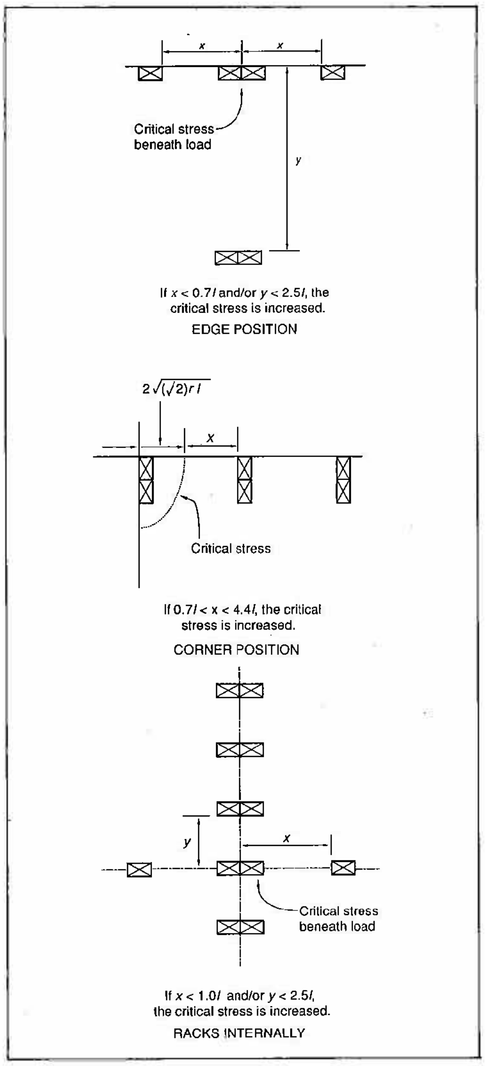

Fig 2.4 Various positions of maximum stress for different combinations of loadings (C & CA)

Table 2.8 **Values of Radius of Relative Stiffness (l) in Millimetres (C & CA)**

Depth of Slab (mm)	Modulus of Subgrade Reaction (MN/m³)	Grade of Concrete (N/mm²) 20	30	40
150	14	850	880	900
	27	720	740	760
	54	600	620	640
	82	550	560	580
175	14	960	990	1010
	27	810	830	850
	54	680	700	720
	82	610	630	650
200	14	1060	1090	1120
	27	890	920	940
	54	750	770	790
	82	680	700	710
225	14	1160	1190	1220
	27	970	1000	1030
	54	820	840	860
	82	740	760	780
250	14	1250	1290	1320
	27	1050	1080	1110
	54	890	910	930
	82	800	820	840
275	14	1340	1380	1420
	27	1130	1160	1190
	54	950	980	1000
	82	860	880	910
300	14	1430	1480	1510
	27	1210	1240	1270
	54	1020	1040	1070
	82	920	940	970

mining their equivalent radius. With a knowledge of r, the depth of slab and the modulus of subgrade reaction, Table 2.9 can be used to find the tensile stress in the slab due to a 1 tonne load for various positions of that load on the slab. The unit load stress should be multiplied by the total load in tonnes, together with the appropriate load factor to find the design stress. This stress may then be either increased or decreased, depending on the relationship of adjacent point loads.

Various combinations of loads relative to slab joints are shown in Figure 2.4. For the point load under consideration, the distance to each adjacent load is considered in turn and if within the zone of influence a percentage of the stress, derived from Figure 2.5, is either added or subtracted to give the total stress.

For corner and edge slab conditions the total stress can be reduced provided that the slabs are adequately dowelled together. A 15% stress reduction can be applied for edge conditions and a 30% reduction for corners. The final stress should be compared with the flexural strength and if exceeded the calculation should be repeated for either an increased concrete strength or an increased slab depth. This method of analysis is amenable to computer interpretation and during the next few months a program will be developed making the design of industrial floor slabs an even easier and faster process.

John Morrison

Table 2.9 **Values of Tensile Stress in kN/M^2 for a Loading of 1 Tonne (C & CA)**

Depth of Slab	CBR	Modulus of subgrade reaction	Loading Position on Slab: Corner					Edge					Internal				
			Radius of Area of Loading (mm)														
(mm)	(%)	(MN/m^3)	50	100	150	200	250	50	100	150	200	250	50	100	150	200	250
150	< 2	14	1370	1180	1050	940	840	1250	1110	980	880	790	780	680	590	510	450
	3	27	1320	1130	980	860	770	1170	1030	900	800	720	740	640	540	470	410
	10	54	1280	1070	910	790	690	1100	960	830	730	640	700	600	500	430	370
	30	82	1250	1030	870	740	640	1050	910	780	680	600	680	570	480	400	340
175	< 2	14	1020	890	800	720	650	930	840	760	680	620	580	510	450	400	350
	3	27	990	850	750	670	600	870	790	700	630	570	550	480	420	370	320
	10	54	960	810	700	620	540	820	740	650	570	510	520	450	390	340	290
	30	82	940	790	670	580	510	790	700	601	540	480	500	440	370	320	280
200	< 2	14	790	700	630	570	520	720	660	600	550	510	440	400	360	320	290
	3	27	770	670	600	540	480	680	620	560	510	460	420	380	340	300	260
	10	54	750	640	560	500	440	630	580	520	470	420	400	360	310	270	240
	30	82	740	620	540	470	420	610	550	490	440	400	380	340	300	260	220
225	< 2	14	640	560	510	470	430	570	530	490	450	420	350	320	290	260	240
	3	27	620	540	490	440	400	540	500	460	420	380	330	300	270	240	220
	10	54	600	520	460	410	370	500	470	420	390	350	310	290	250	230	200
	30	82	590	500	440	390	350	490	450	410	370	330	300	270	240	210	190
250	< 2	14	520	460	420	390	360	470	440	410	380	350	280	260	240	220	200
	3	27	510	450	400	370	340	440	410	380	350	330	270	250	230	200	180
	10	54	490	430	380	340	310	410	390	350	320	300	250	230	210	190	170
	30	82	490	420	370	330	290	400	370	340	310	280	240	230	200	180	160
275	< 2	14	430	390	360	330	310	390	370	340	320	300	230	220	200	190	170
	3	27	420	380	340	310	290	360	340	320	300	280	220	210	190	170	160
	10	54	410	360	320	290	260	340	320	300	280	260	210	200	180	160	150
	30	82	410	350	310	280	250	330	310	290	260	240	200	190	170	150	140
300	< 2	14	370	330	300	280	260	330	310	290	280	260	200	190	170	160	150
	3	27	360	320	290	270	250	310	290	280	260	240	190	180	160	150	140
	10	54	350	310	280	250	230	290	270	260	240	220	170	160	150	140	130
	30	82	350	300	270	240	220	280	260	250	230	210	170	160	150	130	120

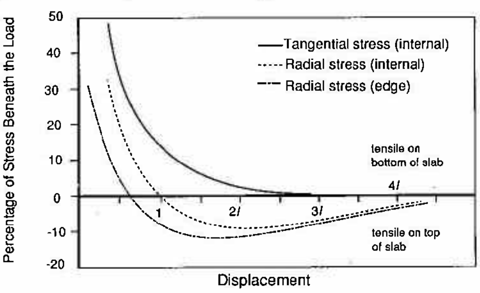

Fig 2.5 Additional stress due to an adjacent load (C & CA)

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2.4 C & CA
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'Concrete Ground Floors': R C Deacon

2.6 BCA
Interim Technical Note 11: 'The Design of Ground-Supported Concrete Industrial Floor Slabs': J W E Chandler and F R Neal April 1988

New British Embassy and Housing, Riyadh

Project Data

Client	Foreign and Commonwealth Office
Architects	Trevor Dannatt and Partners
Civil and Structural Engineers	Buro Happold
Services Engineers	Dale and Goldfinger (M & E)
Quantity Surveyors	Gardiner and Theobald
Contractors	Laing Wimpey Alireza
Date completed	1985

Until 1981 the Saudi Arabian Ministry of Foreign Affairs and all the national Embassies were located in Jeddah, some 600 miles south east of Riyadh, the capital and seat of the Saudi Government. At that time a decision was made that the Ministry and all the Embassies should transfer to a new Diplomatic Quarter (Fig 3.1) to be located on the northern outskirts of Riyadh.

The British Government, through the Overseas Estates Department of the Foreign and Commonwealth Office, selected Trevor Dannatt and Partners as architects, with Buro Happold as consulting civil and structural engineers, to design the new British Embassy and staff housing. Our brief included all structural work for buildings and external works, as well as potable and irrigation water supply, site drainage and internal drainage. Design work started in 1981 and the buildings were completed in 1985.

Layout and Location of Site

The 10.75 hectare site for the offices, Ambassador's residence and all the amenity facilities was bounded by a major boulevard to the south and by a secondary road to the west containing all the infrastructure services. The main entrance and the office building front onto the boulevard (Fig 3.2). The residence is at the opposite end of the site and in between lie the service and amenity buildings. Housing is located north and north west of the Embassy, close to the Wadi Hanifeh.

The site is bounded on three sides by high reinforced concrete framed limestone walls, and by open fencing to the boulevard. The open spaces within the site are intensively landscaped with gardens, swimming pools, a tennis court and water features. Car parking and walkways are all protected from the sun by timber kafess structures.

Foundations for all major structures on the main site are reinforced concrete bases bearing on the rock, which was found to be competent within 1m of ground level. All foundation positions were probed to a depth of 5m to ensure there were no cavities within the immediate zone of the site, especially along the Wadi Hanifeh, which runs to the north.

Office Building

The 3 storey office building is symmetrical about a central stair to the two upper floors. A colonnade extends along the front of the building. The exterior is faced with beige Riyadh limestone with horizontal bands of local granite (Fig 3.3). The brief called for individual offices rather than open plan areas and excluded the use of suspended ceilings. The space requirement of the offices varied considerably. To meet this, a concrete coffered floor slab system with wide beams on a 3.2m grid, was developed

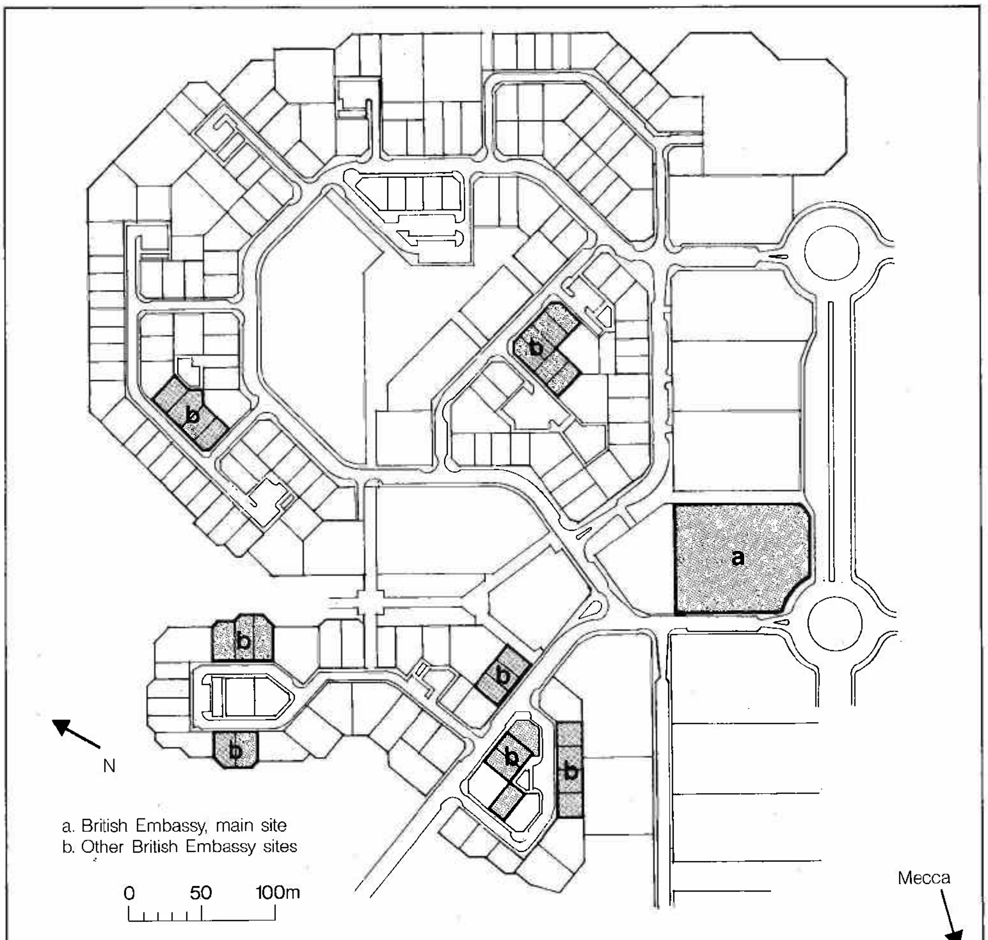


Fig 3.1 Part plan of Diplomatic Quarter showing Boulevard and British Embassy main site to right (shaded)

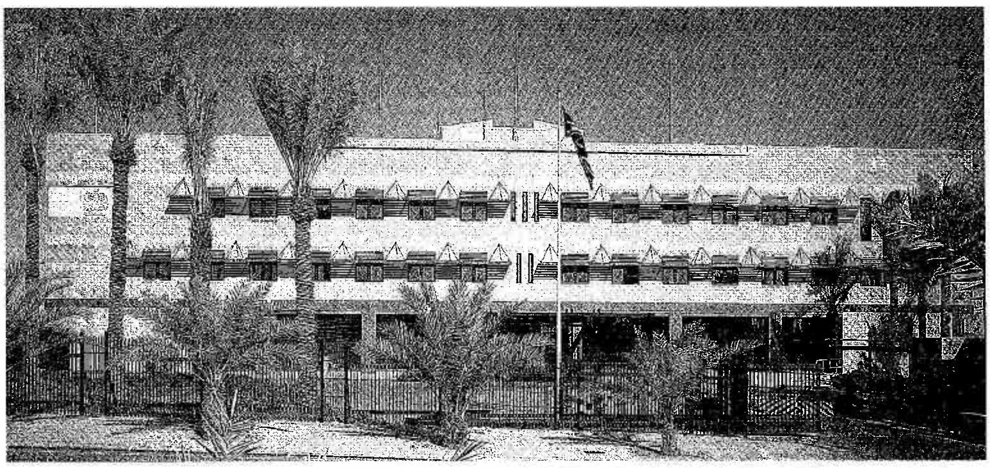
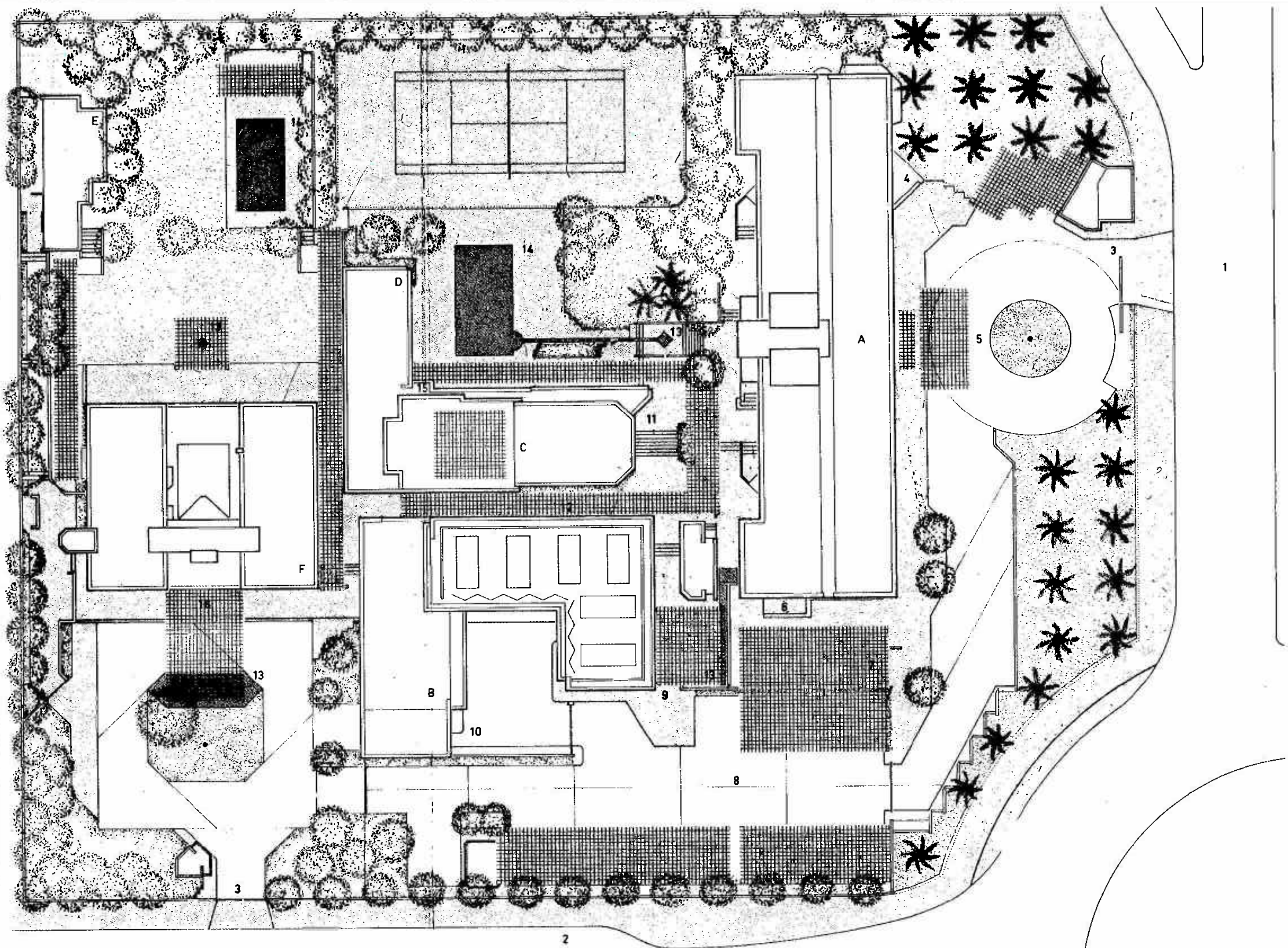


Fig 3.3 South front of office building, across Boulevard



Main site general plan

A. Offices
B. Service building
C. Club & hall
D. Squash
E. Residence staff
F. Residence

1. Boulevard
2. Feeder road
3. Entrance gates
4. Commercial entrance
5. Offices entrance
6. Visas entrance
7. Visa court
8. Service drive & parking
9. Amenities entrance
10. Service yard
11. Hall entrance
12. "Street"
13. Water features
14. Pools
15. Club entrance
16. Residence entrance

N

0 5 10 15 20M

Fig 3.2 Main site, British Embassy buildings and housing

with the architect. The wide beams provide a level soffit to which the block work office partitions can be sealed, the width of the beams allowing the width of the offices to vary. The coffered shutters were purpose made in G.R.P. by a UK fabricator and shipped to Riyadh.

The octagonal columns allowed the block partitions to be built up to them. They were left as cast in a reddish-brown coloured concrete. The ceilings were also left as cast and painted white. The services are therefore exposed, with circular air distribution ducts at high level creating an elegant feature in the central circulation area. The thick external cavity walls are well insulated with deep window reveals. Good quality materials are used throughout with specially designed fan coil units in the window recesses, and special light fittings. Floors in the public areas are travertine and the offices are carpeted.

Ambassador's Residence

Like the office building, the two storey residence is symmetrical about the entrance. From the road the building is formal in character with no domestic windows featured. On the garden side the building

Fig 3.4 Garden elevation of Ambassador's residence

Fig 3.6 Garden Court and terrace of a larger house

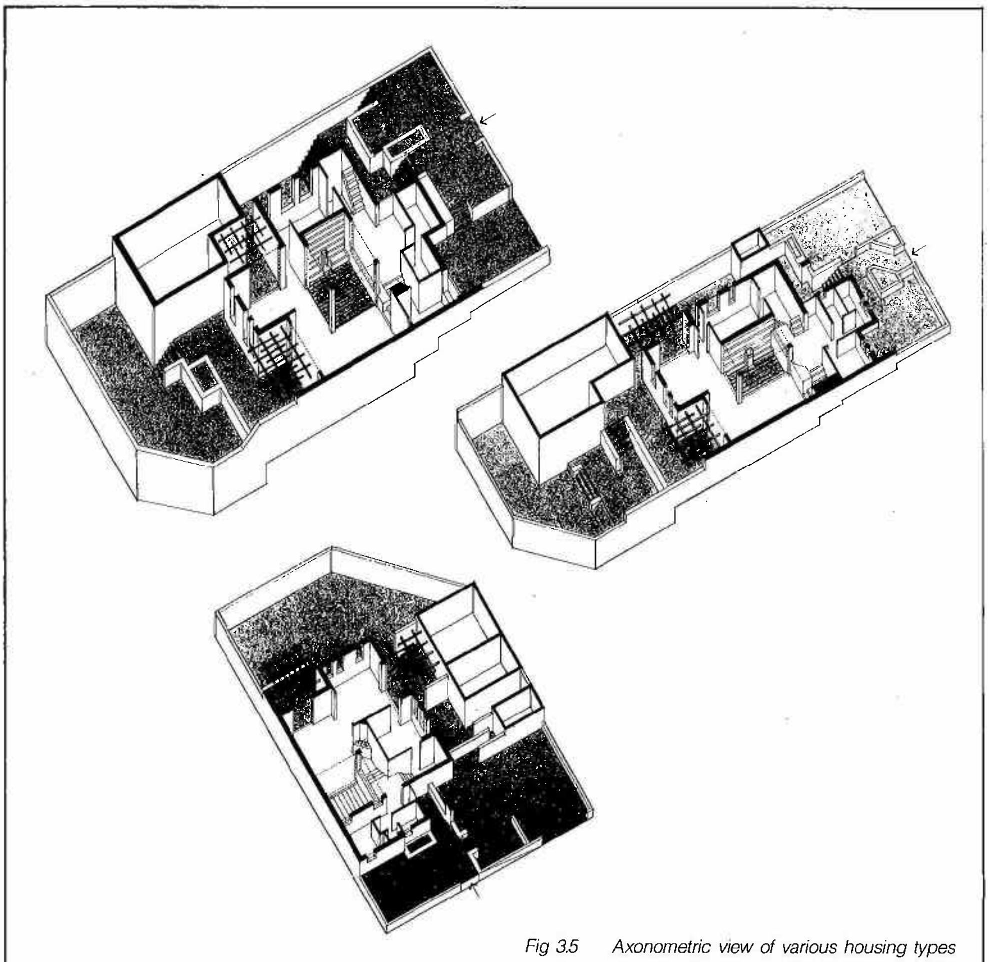


Fig 3.5 Axonometric view of various housing types

is more open with recessed terraces at ground and upper level and a central open space at first floor which articulates the building (Fig 3.4). The expressed concrete frame structure is clad in Saudi granite with infill of beige Riyadh limestone.

Staff Housing

Thirty five houses for the Embassy staff are constructed on seven separate sites, all within close reach of the main office site. The sites vary in plan and topography and some slope steeply towards the Wadi, resulting in a great variety of foundation solutions and details. Although the buildings are various in form, a common identity is established in the arrangement of rooms around internal courtyards, and in the location of entrance hall and stair (Fig 3.5).

The load bearing walls and internal courtyard frames for the houses are founded on reinforced strip footings which, when ground levels dictated, were formed on built-up compacted fill. The planning requirements demanded a high degree of privacy and all the houses were bounded by high screen walls (Fig 3.6). The external block walls of the houses are finished with a beige tyrolean render and Riyadh limestone is used for plinths and special features. Hanging timber lattice screens to balconies are a distinctive feature in all of the houses providing privacy and shading.

Ian Liddell

Ocean Village – Regeneration of Princess Alexandra Dock, Southampton

Remedial Works to Quay Walls

Project Data

Client	RAPD
Civil Engineers	Buro Happold
Contractor	Dean & Dyball, Poole
Value	£86,000
Date	Completed September 1986

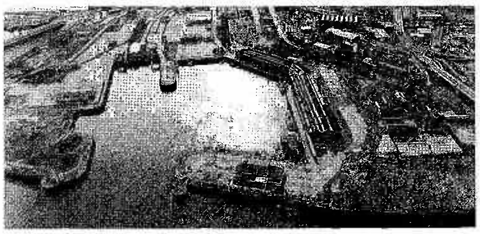


Fig 4.1 Aerial view, from east, of undeveloped Princess Alexandra Dock, Southampton

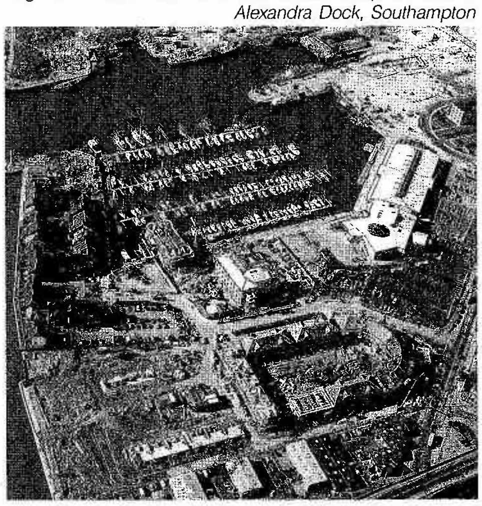


Fig 4.2a Aerial view, from north, of new Ocean Village development

In 1985 Shearwater Property Holdings, a member of the Rosehaugh Group, formed with Associated British Ports a joint company called Rosehaugh Associated Ports Development (RAPD) to develop the 65 acre Princess Alexandra Dock area at Southampton (Fig 4.1). This was to be Shearwater's first 'festival' project, whereby activity was re-generated, bringing the public back to an area of the docks from which it had hitherto been excluded (Fig 4.2a, b), and creating a mix of housing, office development, retail and speciality 'fun and fashion' shopping adjacent to a new marina where water events could also be held.

The existing site contained only a few buildings worth retaining, and a quay wall onto the harbour that had always been potentially unstable and required remedial measures in order to be suitable for the marina. Further, the existing infrastructure of roads, drainage and utilities required upgrading, addition and some replacement to match the demand of the proposed masterplan. Housing was to be provided at the south of Channel Way and by the marina building. A later problem arose in the interpretation of loading limits for quayside railings.

Bank House and Ironside House were refurbished for office accommodation and the 5 storey, Victorian Enterprise House was also converted for office use. An entirely new building, Canute's Pavilion, has been constructed to house speciality shopping, restaurants and pubs. Design was commenced on the £3.0m first phase of this building in October 1985, and it was opened for trading seven months later in June 1986. Phase 2 was completed in October 1987.

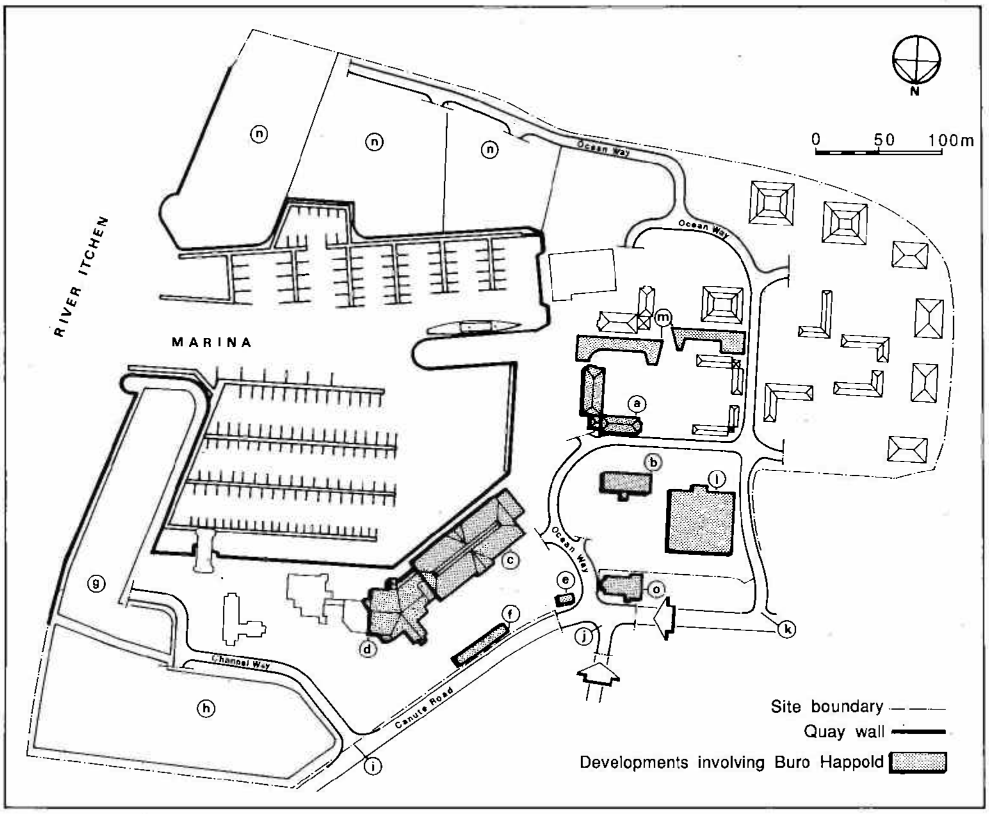
Currently, construction is also under way on a five screen Multiplex cinema for Cannon Cinemas, and a proposed four storey office building, Savannah House, is out to tender for start on site in September 1988.

Remedial Works to Quay Walls

The quay walls to the Princess Alexandra Dock, consist partly of masonry built in Victorian times and partly of sheet piles installed over the last 50 years. At the start of the project, Buro Happold was commissioned to assess the condition and stability of these.

Research in Institution of Civil Engineers material and other published papers indicated that the masonry walls had suffered considerable problems during construction, as illustrated by this extract:

> "The walls had not, however, been carried to their full height when some heavy slips of earth caused some portions to move forward ... The water saturating the soil behind the walls caused a greater pressure than before, particularly as the tide receded, and the walls still partially moved forward. Upon this, additional engineering advice was obtained, and a plan for attaching land-ties to the walls



a Savannah House
b Enterprise House
c Canutes Pavilion
d Canutes Pavilion, Phase II
e Bank House
f Ironside House
g Quay Housing
h Channel Way Housing
i Dock Gate 1
j Dock Gate 2
k Dock Gate 3
l Multiplex Cinema
m Water Feature
n Southern Residential Area
o Ferry House

Fig 4.2b Ocean Village site plan

Infrastructure Design and Refurbishment

Project Data

Client	RAPD
Civil Engineers	Buro Happold
Contractor	Machola Ltd (Channel Way) Dean & Dyball Const. Ltd (Ocean Way)
Value	£212,000 (Channel Way) £1.5m (Ocean Way)
Date	Channel Way completed March 1987 Ocean Way due to be completed October 1988

was suggested ... After an expenditure of about £10,000, the land-ties proved unequal to their task; several of the ties broke, others stretched, and the walls still moved forward. Further mischief was, however, prevented, by removing the soil from the back, and thus relieving the walls of as much weight as possible ..."

ICE Proceedings 1858

Stability calculations indicated that existing factors of safety against sliding were insufficient. Diving inspections showed that mortar joints to the granite blocks had in many areas suffered severe erosion. Jointing to the brickwork which made up the face of the walls below the water line was in many places in very poor condition, and in some areas brick work was missing to depths of 600mm (Fig 4.3a, b).

A programme of remedial works to the walls was established with two principle aims. Firstly to repair jointing in the walls, and replace damaged or missing brickwork; and further to deposite suitable material in front of the toes of walls to provide

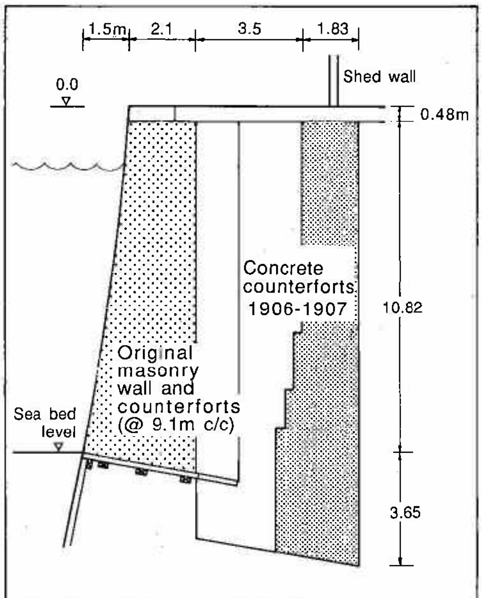


Fig 4.3a Typical quay wall profile

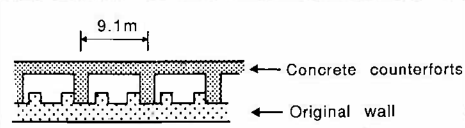


Fig 4.3b Typical layout of counterforts

additional passive resistance against sliding. As the proposed marina required much less draft than had been necessary when the quays were used for the roll-on roll-off car ferries to France, tipping was an attractive option.

Dean & Dyball of Poole won the tender for the remedial works, at a price of £86,000 and the work was carried out in the summer of 1986, under ICE (5th Edition) Conditions of Contract. It was supervised by Buro Happold and completed within budget by September 1986.

Sheet Pile Investigation

A diving survey was carried out to all the sheet pile wharves which form the majority of the southern half of the marina. The survey included comprehensive investigation of sheet pile thicknesses using a sea probe. Readings enabled corrosion of the piles to be ascertained and an assessment made of the remaining useful life of the piles. This was generally found to be greater than 40 years and little remedial work was required.

Infrastructure Design and Refurbishment

RAPD had initially hoped to re-use at least part of the existing dock infrastructure – perhaps water mains, drains, electrical supply, and to make use of the concrete and asphalt surfacing over much of the site. Buro Happold was commissioned to report on the infrastructure requirements of the Ocean Village development, using the existing infrastructure where possible, even if only in the short term, and allowing for a staged development throughout the site. A report on this aspect of the development was presented in April 1986 and has been the basis of infrastructure implementation to this day.

The development required new vehicle and pedestrian routes, and links to the city transport systems, a network for surface water drainage to the river or docks, and foul drainage to the city sewers. Incoming utilities had to be matched to the future demand of the village.

Initially, attempts were made to utilise the existing systems but these were fraught with difficulties. The high voltage electrical network was at the end of its life. The routes of many of the drains could not be traced and some of the water mains were so fragile that they cracked even without the presence of any major works. In all but a few minor cases the re-use of existing infrastructure was abandoned at an early stage.

Construction of Channel Way and Ocean Way

The first works from Dock Gate 1 for housing at the marina and Alexandra Quay, known as Channel Way, comprised a 160m road, 7m wide with footways and verges either side. Surface and foul water drainage systems were installed, with primary utilities of HV electrical supply, gas, water and telecom beneath the footway. All works have been executed to adoptable standards in liaison with officers of Southampton City.

An ICE (5th Edition) contract was placed with Machola Ltd late in 1986, and completed satisfactorily in March 1987 at a final value of some £212,000.

Early in 1987 the client, RAPD, then requested the preparation of designs for the Ocean Way infrastructure. These works are, in concept, the same as Channel Way but are on a significantly larger scale, comprising road works from Dock Gates 2 and 3 southwards into the site, with a total length exceeding 1km (Fig 4.2b). Reports have been prepared advising the client of cost contributions to the offsite highway works in Canute Road and beyond, on matters relating to security for the overall development and to possible multistorey car parking to ease land problems.

For surface water drainage of the area, two existing outfalls have been utilised, discharging into the old dock, which is now to be refurbished as a marina.

Foul water drainage discharges into the sewer in Canute Road via Dock Gates 2 and 3. This drainage is sized to serve commercial and light industrial development planned for the site to the west of the marina, and for housing to the south of the marina. A pumping chamber and rising main were required for the sewer from the southern area of the site. Primary utilities are provided as for Channel Way. Extensive crossings beneath the road allow servicing to the sites at each side, and provision is made for data transmission and fibre optics in a system of additional ducts.

A traffic signalling system, linked to the master 'Scoot' system for Southampton City, is being installed at Dock Gate 2 and 3 junctions to Canute Road, while an automatic counter system is also provided at each dock gate entrance. This enables the estate management in Canute's Pavilion to monitor traffic entering and leaving Ocean Village.

This contract, by Dean & Dyball Construction Ltd, commenced in June 1987. Together with some additional works and modifications instructed by the client it is nearly completed at an estimated cost of £1.5m.

For both Channel Way and Ocean Way, road

Refurbishment of Bank, Ironside and Enterprise Houses

Project Data

Client	RAPD
Architects	Covell Matthews Histon Partnership
Civil Engineers	Buro Happold
Services Engineers	Buro Happold
Quantity Surveyors	Norveil and Partners
Contractor	Conder Projects Ltd
Value	Bank House £50,000 Ironside House £110,000 Enterprise House £1.2m
Date	Bank House completed September 1986 Ironside House completed February 1987 Enterprise House completed March 1988

construction has typically been over existing areas of concrete slab construction laid down for the docks' previous use. Wherever possible the road construction has been in the form of an overlay, so maximising usage of this existing concrete slab, and avoiding removal merely to replace with full depth pavement construction. Typical construction has been a dense bitumen macadam regulating course/road base, bitumen and macadam base course, with hot rolled asphalt wearing course (Fig 4.4).

Finishes to external works in Ocean Way were specified at a high standard to include block and concrete paving slab footways with pedestrian crossings and routes defined by coloured block-work, lower height HPS colour-corrected street lighting, and an integrated signing system carrying the Ocean Village motif.

Drainage works have been straightforward except for obstructions from old foundations and quay walls and, in the deeper trenches, the tidal effects on the water table. The construction of the roads and drainage works has also had to take account of the client's requirements to use certain areas for temporary car parking, exhibitions and other activities connected with the promotion of Ocean Village. This need for coordination has at times slowed construction progress.

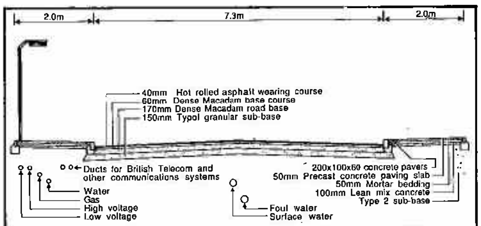


Fig 4.4 Typical cross-section through Ocean Way

Refurbishment of Bank, Ironside and Enterprise Houses

Two of the original site buildings, Bank House and Ironside House, were refurbished during the first phase of work on Ocean Village. Both buildings required small scale commercial services installations with emphasis on the quality of the visible elements, such as radiators, uplighters and other fittings. As these buildings were on the Canute Road, utility connections did not present difficulties even at a time when the site services infrastructure was still in the preliminary stage. The structural work necessary was minimal. Analysis showed that Ironside House required a new portal frame to provide an adequate amount of safety for stability. Apart from this only minor beams and lintels were required (Fig 4.5).

Fig 4.5 Ironside House, a view of the interior

Fig 4.6 Exterior of refurbished Enterprise House

Next followed the refurbishment of a 5 storey basement Victorian warehouse, Enterprise House, into speculative offices (Fig 4.6). The principal features of the building are massive brick walls, low ceiling heights with large exposed timber beams, and a far from watertight basement. There are large timber trusses spanning the full width of the building at the top, supporting the roof. The only new construction involved a central access stairtower including two lifts and a staircase. The building was fully surveyed and analysed to check its stability and to detail repairs to the fabric where necessary.

An early design exercise compared the benefits of a single heating plant located centrally in the basement, heating costs being included in a services charge to tenants, against heating from small gas-fired boilers on each floor. The latter was chosen due to the problems of water leakage into the basement, and the much lower estimated capital costs. This decision was later justified by the tender prices. The final scheme involved two pressurised systems per floor with separate gas metering to each.

Electrical distribution to the building is provided by tenants distribution boards and sub-metering on each floor, linked to 3 compartment floor outlet boxes in the existing timber floors. Data and telecom compartments are provided. Although ceiling heights are low, the client's hope for an uplighting scheme was realised by the use of freestanding 250W Son-T uplighters. These are powered from an additional socket in the floor outlet boxes, controlled and switched via central contractors. A bank of small downlighters using 7W PL lamps is located in the ceilings to give a low background lighting level for general access and circulation purposes.

Canute's Pavilion – Phases 1 and 2

Fig 4.7 Central mall of Canute's Pavilion, Phase 2

The construction of Canute's Pavilion Phase 1 has provided approximately 6700m^2 of mixed retail and studio office accommodation on two floors surrounding a central mall (Fig 4.7). The quayside sheds Nos. 7 and 8 were demolished to allow the construction of Phase 1 of Canute's Pavilion. The new building adjoins a reminder of the site's former use, the existing brick-built 'Continental Booking Office' which was converted to form retail accommodation and the new main entrance.

It was originally intended to found the new structure on the existing ground beams and piles. However, despite the fact that there were no signs of distress within the existing masonry walls, excavation close to a number of piles indicated that these had been of timber, in many cases were rotten, and in some cases non-existent. A piling sub-contract was let to Cementation who, in a period of two weeks, installed some 130 continuous flight auger piles 14m long. Each pile location was pre-probed to a depth of 5m to detect any obstructions from old foundations and services which could be expected in such a redevelopment site. A number of obstructions were found and these were cleared in time to allow continuous operation of the piling rig.

The superstructure of Canute's Pavilion is a steel portal frame propped at the centre with 'Y' shaped columns, and with a first floor construction running down each side of the building. The structure was

Canute's Pavilion

Project Data

Client	RAPD
Architects	Covell Matthews Histon Partnership
Civil, Structural, Services and Fire Engineers	Buro Happold
Quantity Surveyors	Norveil and Partners
Contractor	Conder Projects Ltd
Value	Phase 1 £3.0m Phase 2 £3.15m
Date	Phase 1 completed June 1986 Phase 2 completed October 1987

Multiplex Cinema

Project Data

Client
Architects
Structural Engineers
Services Engineers
Quantity Surveyors
Contractor
Value
Date

originally conceived to distribute loads on the existing foundations and, although these were not eventually used, the form remains. The steel structure was designed in mild steel to minimise any delays in the supply of steel for fabrication, and the first floor was formed using extruded pre-cast pre-stressed planks, being a construction familiar to the contractor.

Buro Happold were further appointed to produce a performance specification and schematic design for the services at Canute's Pavilion. This comprised provision of utilities to retail units, heating, lighting, and a ducted ventilation system with gas fired air handling units for the mall, general power and lighting to the studio offices, together with a comprehensive fire alarm, detection and smoke control system. The final account for the mechanical and electrical services sub-contracts, carried out by I.E.I. Southern Limited, totalled £490,000.

Electrical distribution to the retail units and studio offices is provided by means of a bus-bar system at high level on the ground floor. Tap off boxes are located as required along the length of the bus-bar, to serve both ground and first floor units. The premises are metered centrally by the SEB on the M1 Demand Tariff with sub-metering in each unit administered by the centre management. Currently, the maximum demand recorded by the metering equipment is in the order of 220–230kVA. Power is provided to the premises by a new sub-station located in the north car park, adjoining the diesel standby generator which serves the disabled/escape lift in the north lobby.

A small number of specialist food units and a wine bar/bistro were located at first floor level. These required provision of a greater number of services than the normal retail units, including gas supplies and mechanical extract ventilation from kitchen areas.

The fire engineering of the building presented the most challenging part of the whole design. From the outset it was intended that all retail units, studio offices and circulation areas would be fully sprinklered. The mall areas would not however be sprinklered, provided that no combustible materials were to be present. These proposals were accepted by the authorities. Each unit is provided with call points and sounders linked to the main zone panel. The smoke control philosophy is that all shop units vent into the two-storey mall, up into one of two high level reservoirs from which smoke is exhausted by automatic vents (Fig 4.8a, b). Offices are separated from the mall by fire resistant construction, and vent directly to the outside.

Phase 2 of Canute's Pavilion is a two-storey addition of some 3500m². The design of the structure is more sophisticated than that of Phase 1 and uses grade 50 steel, and composite metal deck construction with in-situ concrete toppings. Stability is provided by cross-bracing. The result, although a more complex geometrical structure, is more slender and elegant than Phase 1 (Fig 4.9). The superstructure is again founded on ground beams, and continuous flight auger piles in this phase were installed by Stent Construction. The main contractor was Conder Projects Ltd.

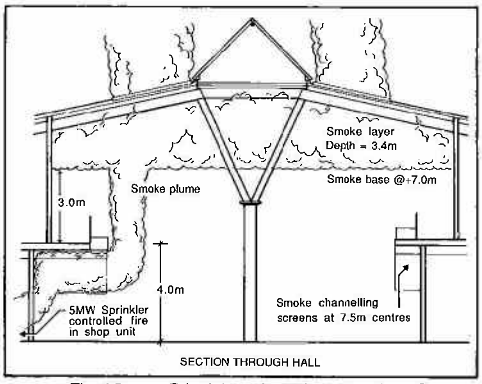


Fig 4.8a Principles of smoke extraction, Canute's Pavilion, Phase 1

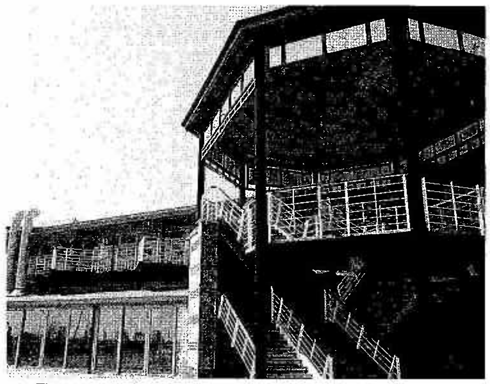


Fig 4.9 External stairways of south octagon, Phase 2, Canute's Pavilion

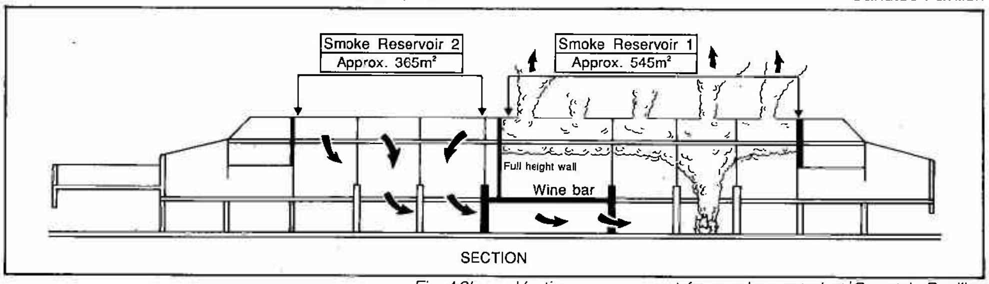


Fig 4.8b Venting arrangement for smoke control at Canute's Pavilion

Services are much in line with Phase 1 but with higher specification of fixtures and fittings throughout. All fire alarm systems and sprinkler installations are linked to the Phase 1 control position.

Further Development of the Site

Savannah House, 2500m² in area, and 3 storeys high, is the first of the new office buildings on the site. Due for completion in June 1989, the structure follows in principle that of Canute's Pavilion Phase 2. The building straddles a massive quay wall of one of the now filled docks. Foundations have been designed to avoid both this obstruction and those caused by the foundations to an old quayside warehouse. Features of the building include an external brise-soleil, a full access raised floor system for electrical data and BT distribution, free-standing uplighters, perimeter heating and full mechanical ventilation, with the option of conversion to comfort cooling at a later stage.

The Cannon Multiplex 5 screen cinema is approximately 50m² square on the ground, and 11 metres high. Architects, quantity surveyors, and service engineers for the complex are Howard and Unick of Glasgow. The whole of the super structure is supported on a minimal arrangement of ground beams on piles. Due to the proximity of the structure to Enterprise House and concern over vibrations from driven piles, continuous flight auger piling was again used for this structure, installed by West Piling. The main contract is let to Dean & Dyball Construction, with completion due in mid 1989.

To minimise wasted space in the construction, and to ease the detailing of the acoustic separation between the cinemas, the structural steel solution is a hybrid propped frame with cross bracing in all four walls, and in the roof. This has allowed the columns to be no larger than required as cladding rails, restrained top and bottom to take the windloads within the walls. There is a two storey construction down the centre of the building. The floor of this structure, which requires two hour fire protection, is formed of an in-situ rib slab spanning nine metres onto concrete encased steel beams and steel columns.

Cannon Cinemas Ltd
Howard and Unick, Glasgow
Buro Happold
Howard and Unick
Howard and Unick
Dean & Dyball Construction Ltd
£2.5m
Due for completion mid 1989

Savannah House

Project Data

Client RAPD
Architects Covell Matthews Histon Partnership
Structural Engineers Buro Happold
Services Engineers Buro Happold
Quantity Surveyors Norvell and Partners
Contractor Currently out to tender
Date Due for completion June 1989

Quayside Railings – What loadings can the public exert?

In accordance with BS6399, the railings to the quayside were designed for a load of 0.74kN/m at a height of 1.1m above the ground in order to reduce the risk of people falling into the harbour. However, RAPD had successfully arranged events at Ocean Village with attendances of up to 40,000 people. One day it occurred to them to ask how many rows of people these railings could safely restrain!!

Various codes dealing with loadings on railings were consulted. This led to some confusion as there is no code for dockside railings. The buildings code suggests a maximum load of 0.74kN/m at 1.1m above ground. The highway barriers code suggests 1.4kN/m at 1.05m, and the codes for barriers in stadia suggest various forces different from the above, dependant on crowd density, slope and activity (Refs 4.1, 4.2, 4.3). However, none of the codes gives the reasons for these loads.

From data assembled by the Institution of Structural Engineers following the Popplewell Commission on Safety of Sports Grounds, it was feasible to determine that the current codes and standards stem essentially from tests done for the chief structural engineer of the Ministry of Works in 1946 (Ref 4.4). These tests were carried out on a barrier 3′6″ (1.07m) above the ground using "male building operatives in rows 1–11 deep in 57 different combinations" to measure the effect of 'lean', 'strain' and 'heave' force on the barrier as defined below:

'Lean Force': The horizontal force recorded by a tightly packed crowd with persons bearing one on another, but not having any special interest ahead.

'Strain Force': The horizontal force applied in similar circumstances but at a moment when the crowd exerts an additional pressure on the barrier in its efforts to see items of special interest ahead.

'Heave Force': The horizontal force exerted in similar circumstances by a crowd in panic.

The men were found to stand 470mm apart and each row was 340mm deep. A factor of 0.875 was used to allow for women and children in the crowd. The lean force was measured as 0.18kN/m width per row. The heave force was measured as 2 × the lean force, but a factor of 1.6 was chosen to allow for factors of safety and short term stresses. From this a crowd force was established of 0.73kN/m depth /m width of crowd. In other words, three rows of people is approximately equivalent to

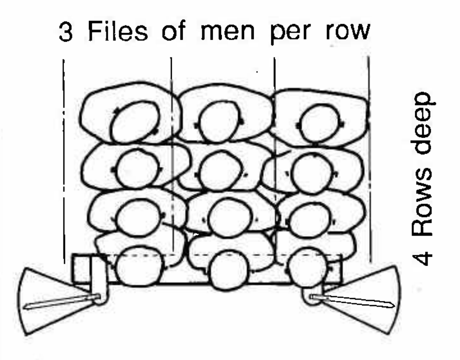


Fig 4.10a Load test on barriers

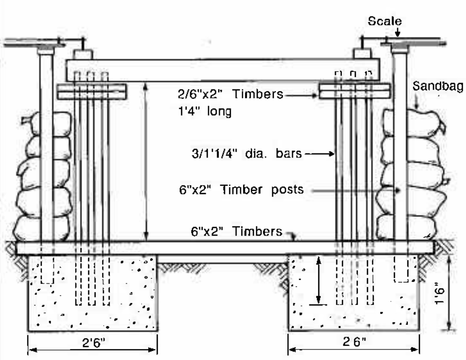


Fig 4.10b Elevation of test barrier

0.75kN/m along the railings (Fig 4.10). As a result of this evidence it was decided to restrict events involving large numbers of people, unless special crowd barriers were erected along the edge of the quays.

Ocean Village – A Success

Now that construction of all the initial elements is complete, Ocean Village, under RAPD auspices, has started to establish a national reputation, drawing people from near and far to its events, shops and restaurants.

Enterprise House is now let and the demand for housing continues. Regeneration of many of the areas outside the immediate boundary of Ocean Village is well underway, with Ocean Way opening up the southern and eastern parts of the site for offices, industrial units, a museum and further housing development.

Rod Macdonald, Vincent Grant, Robin Clark and Peter Moseley

References

4.1 BS 6399: Part 1: 1984 "Loading for buildings: Code of practice for dead and imposed loads."

4.2 Department of Transport (Highways & Traffic Division) Technical Memorandum BE5 "The Design of Highway Bridge Parapets – 4th Edit" 1986.

4.3 BS 3049: 1976 "Specification. Pedestrian guard rails (metal)".

4.4 Ministry of Works. May 1946. "Experimental crush barrier tests".

The Theatre as a Built Form

Theatre Royal, Bath

Project Data

Client	Theatre Royal (Bath) Ltd – Chairman: Jeremy Fry
Architect	Dowton & Hurst – Donald Armstrong
Designer	Carl Toms
Structural Engineers	Buro Happold
Services Engineers	Buro Happold (later phases only)
Quantity Surveyors	Gleeds
Contractor	Longs
Steel Sub-contractor	Curtis Steel, Frome
Date	1982
Cost	£2.0m

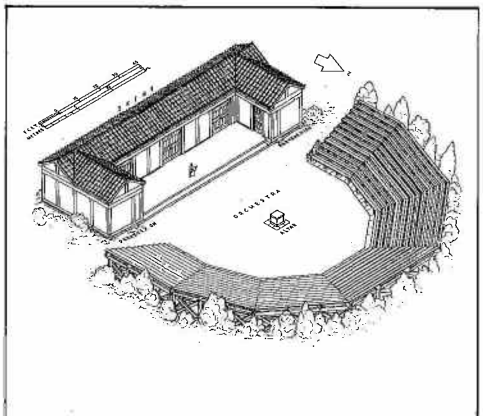


Fig 5.1 Early theatre in Greece — (Ref R + H Leacroft — "Theatre and Playhouse" 1984)

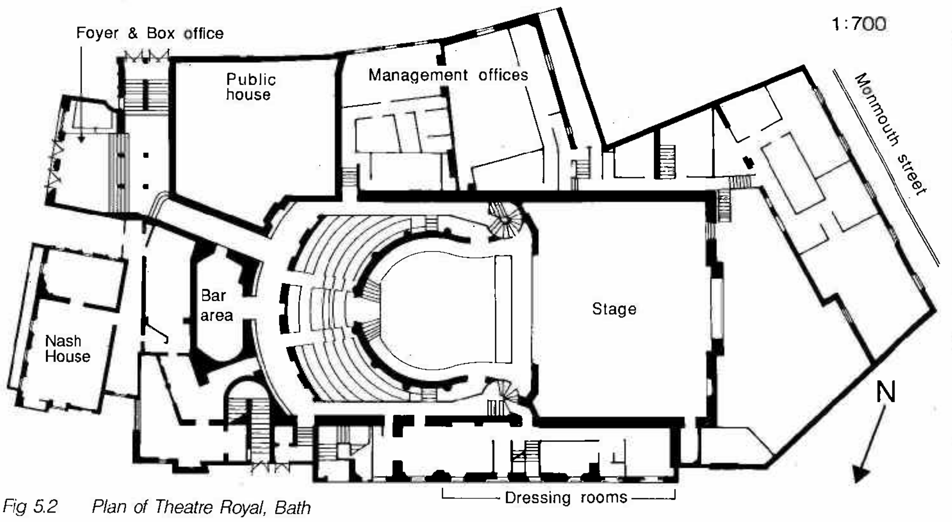


Fig 5.2 Plan of Theatre Royal, Bath

In ancient Greece, the space adjacent to the market place, or 'Agora', became known as the 'orchestra' and gradually evolved into a focus between actor, chorus and audience. The audience sat on timber benches but these were later superseded by steeply tiered stone fan-shape seating giving shorter sight lines and better acoustics. Then followed the 'skene' to frame the performance space and thereafter the 'proskeneum' (Fig 5.1).

In Europe, too, built-forms have been evolving that are specific to the theatre environment. Diverse forms such as 'courtyard', 'proscenium', 'picture stage', 'full thrust', 'semi thrust', and 'in the round' have all been part of the aim to improve communication between performer and audience (Ref 5.1).

Today it is also important that this built form can be operated, managed and maintained by the minimum of personnel so reducing running costs. From theatre management's point of view, the struggle is always the same — to secure financial viability. In this, the development of the box office, modern ticketing systems, and extended use of foyer and associated cafeteria and bar spaces beyond theatre hours, all play an important part.

Restoration of a Provincial Theatre

The renovation in the early 1980s of the 900-seat Theatre Royal in Bath illustrates many of these aspirations (Fig 5.2). Despite containing some historically important early theatre machinery, the entire Georgian fabric required refurbishment, and the installation of modern management facilities, and new stage and flying facilities.

Under the guidance of architect Donald Armstrong of Dowton and Hurst, and to designs by Carl Toms, the restoration to 'front of house' areas included a new box office, bar facilities and a brasserie at the low level beneath the Dress Circle and Gallery seating. New management facilities were introduced behind the Sawclose facade and new dressing rooms behind the Beaufort Square facade. These changes did not impinge significantly on the existing Georgian fabric but greatly extended the quality and range of activities possible within the Theatre complex.

Architecturally, however, the new fly tower to the enlarged stage area did have considerable impact on the surrounding fabric of the City. A number of devices were considered to lessen the impact of this element on the skyline and, after much discussion, a slightly asymmetrical fly tower shape finally received approval from the Royal Fine Arts Commission. To minimise the resulting building mass, it was to be clad in lead and the delicate Georgian stonework below carefully restored (Fig 5.3).

Fig 5.3 Beaufort St facade, Theatre Royal

Structure of Fly Tower and Stage

The new fly tower and stage area were arranged in order to give a standard working format (Fig 5.4). The huge costs of mounting new shows could then be shared between a number of similar production companies, and tours between such centres would no longer have the added expense of continuous adaption. In order to achieve this, the tower had to stand some 23m above the new 12 × 12m stage and adjustable orchestral pit, and was some 8m higher than the existing ashlar-faced random rubble walls. It also had to provide for flying fifty 1000lbs (400kg) counterweighted bars suspended from the roof steels through the fly grid, and thirteen 6000lbs (250kg) hemp lines suspended directly from the fly grid irons.

Appraisal of the Georgian fabric had identified that the existing stone walls could not support the increased vertical loads of the new fly tower and would not be stable during construction after the

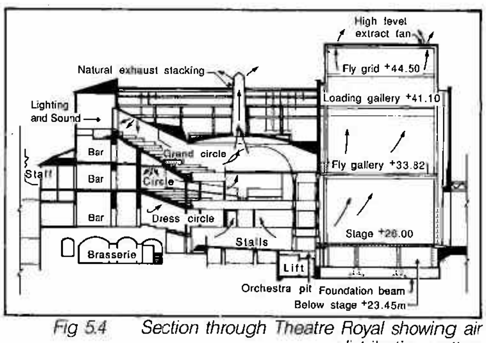


Fig 5.4 Section through Theatre Royal showing air distribution pattern

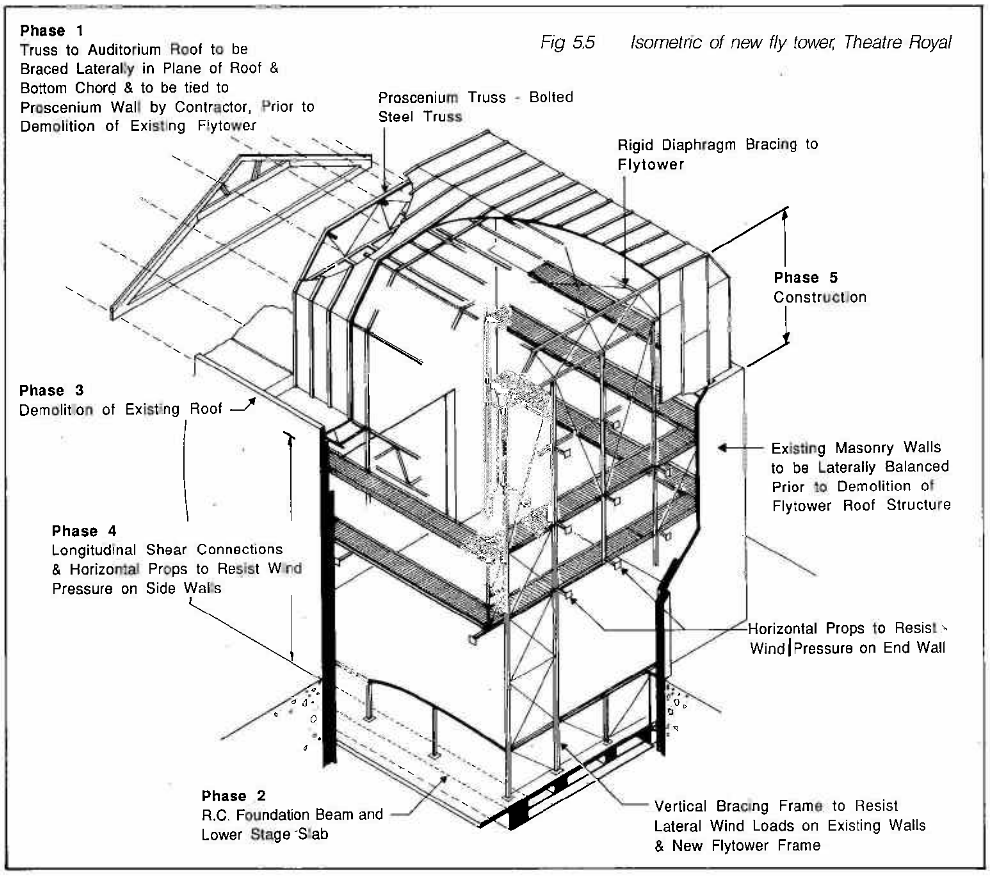


Fig 5.5 Isometric of new fly tower, Theatre Royal

removal of the existing roof to the stage. Although the side walls to the stage and auditorium would stabilise the fly tower in the longitudinal direction, the existing fabric could not resist the additional 150 kN lateral wind loading.

The solution was to insert a new structural steel fly tower as a rigidly braced tube within the existing walls on new foundations, without disturbing the existing stone walls during construction and without undermining the existing rubble stone walls. This proved to be quite a tricky business (Fig 5.5). After demolition of the stage roof and following the first phase of steel construction up to eaves level, the stone walls could then be propped against wind loadings by the U-ring trusses of the fly gallery at ±33.15m level and at eaves level. These were connected by outrigger plates into padstones within the walls themselves. Wind pressure on the rear stage wall and from the full fly tower could then be transferred by these outriggers by way of longitudinal shear connection on to the side walls of the stage and auditorium. The new vertical steel bracing frames between the 203mm Universal steel columns straddling the door at the rear of the stage assured lateral stability.

The vertical loading from the 533 × 210mm Universal beams supporting the roof carcassing, the counterweighted pulley sets and the ten pairs of 254 × 76mm channel sections forming the fly grid at ±44.5m, is then transferred via a new bolted steel truss on to two new 254mm Universal columns located just behind and to either side of the existing proscenium opening at the rear of the stage. It is further supported at the rear stage by the 203mm Universal columns which also form the lateral bracing frames. In order to enhance resistance to damage, the columns were encased in concrete up to fly grid level, and the remainder coated with intumescent paint providing one-hour fire resistance.

Foundations to the new fly tower, 1.5m below the existing wall foundations, are of reinforced concrete beams 1.2m wide × 1.5m deep running front to back. These spread the loads from the proscenium columns and rear bracing frames evenly onto the founding clay strata as "beams on an elastic foundation". They were carefully constructed in pre-determined sections to prevent the undermining of the existing stone walls. All outrigger plates were slotted vertically to allow differential settlement between the new and existing structures.

Environmental Design

During the initial restoration a simple ventilation system was installed as part of a design-and-build package whereby tempered air was supplied from the rear of the seating and extracted from the auditorium through grilles at high level, and through the chandeliers at the centre of the dome ceiling. Heating to the 800m^2 auditorium, comprising Upper Circle, Circle and Dress Circle and having in all 900 seats, was provided by radiators. The 208m^2 of stage with 22m fly tower over it, however, was not heated.

This system, as installed, provided inadequate ventilation and was too noisy to run during performance. Furthermore, it was difficult to maintain and lacked proper control. Buro Happold were asked to advise on measures to improve the control and operation of the system within the limits of an extremely tight budget. To provide a better scientific basis for proposing improvements, temperatures at various parts of the auditorium, with the system off and with 100% occupancy, were measured by hand-held digital thermometers and thermohygrographs (Fig 5.6). Although the temperature rise in the large volume of the stalls was not dramatic, a rise of some 6°C was recorded in the tiered seating. Further, with the system as installed and external air temperature of -1°C, the absence of sufficient heat in the fly tower led to discomfort during rehearsals and a wave of cold air onto the audience immediately after curtain-up (Ref 5.2).

Sound levels measured with the system in operation were 47 dba, some 22–27 dba above desirable levels of 20–25 dba, or four times as loud as necessary. Much of this was due to the absence in the plant of anti-vibration mountings. In order to evaluate remedial measures and in turn to avoid any really unnecessary cost, contractors were appointed to rectify these matters and to re-

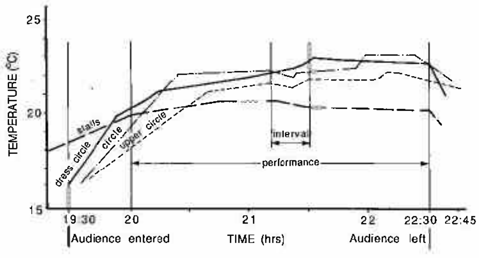


Fig 5.6 Air temperature patterns in Theatre Royal auditorium

West Yorkshire/Leeds City Playhouse

Project Data

RIBA Competition (second place)	
Architect	Derek Walker Associates
Theatre Consultant	John Bury
Structural & Services Engineers	Buro Happold
Quantity Surveyors	Rex Procter & Partners
Date	1985
Estimated Cost	£5.9m

Table 5.1 **Desirable Design Criteria**

Criteria	Summer	Winter
External temperature	28°C dry-bulb 19°C wet-bulb	−1°C dry-bulb −1°C wet-bulb
Internal temperature	21°C to 24°C	19°C to 21°C
Relative humidity	50%–60%	50%–60%
Noise Criteria	NR 20–25	

commission the installations. Considerable improvement was attained but nevertheless the overall performance still falls far short of the desirable criteria for theatres (Table 5.1) (Ref 5.3). However, in view of the minimal remedial expenditure of £7,500 the improvement to the environment was considerable. To fully sustain a successful season of theatre in peak summer months, environmental performance of the auditorium must be matched more closely to the desirable criteria and requires the planned introduction of a full air-conditioning system (Ref 5.4).

The initial refurbishment was deftly undertaken by the main contractor Longs, very ably helped by steelwork sub-contractors Curtis Steel.

Thereafter the plaster and decorative work was beautifully restored to the designs of Carl Toms, and the whole effect completed by a gift of the Chaplin family of new stage curtains suitably monogrammed (Fig 5.7). Together with new front-of-house facilities, new dressing rooms, and new managerial and club facilities, the reconstruction of the Theatre has provided for greatly extended activity, which reflects the changes taking place within the City itself.

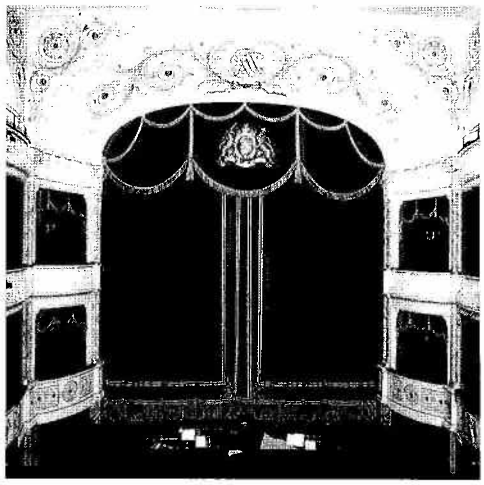


Fig 5.7 Chaplin monogrammed stage curtains

The Perfect Theatre — is there one?

Discussion of the perfect theatre is intense (Ref 5.5). The National Theatre offers three alternatives: the 'thrust stage' of the Olivier, the 'proscenium stage' of the Lyttleton, and the 'studio space' of the Cottesloe. Each offers a space of ideal proportion for a specific type of performance.

Opinions differ as to what in a single auditorium gives the most flexible theatre space. The Derek Walker/John Bury/Buro Happold second-placed competition entry for the replacement of the 'thrust stage' of the existing Leeds Playhouse, within a new theatre complex on a Quarry Hill housing site near the centre of the City of Leeds, is a case in point (Fig 5.8). Our design proposed a 650-person auditorium around a semi-thrust stage together with a 350-seat studio theatre, flexible front-of-house facilities to enable maximum usage and production workshops at the rear (Fig 5.9). To link the theatre to the City, particularly at night, the foyer was placed behind glazed walls beneath a delicate folded steelwork roof reminiscent of the Leeds City arcades and in front of the inevitable bulk of the theatre itself (Fig 5.10).

In order to reduce the direct speech path between performer and semi-thrust stage to 16m, the seating in the main auditorium is located on two raking levels. Blocks of seats are focused on the 8m performance circle at the front of the stage, and the balcony overhang for the second tier of seats is shallow to prevent acoustic shadow below. The stage itself was designed with a fly tower with full grid irons on pairs of 250mm deep channels in order to provide single purchase counterweighted flying facilities throughout the stage. A cranked fire curtain, as at the Barbican Theatre, was then provided to the front of this 20m proscenium with additional lighting bridges over the auditorium, accessible directly from both the rear control lighting bridge at second level and the fly-pass door. This provides a 300 variable patch system with the main lighting positions on the 40 and 60 degree lighting bars above the auditorium (Fig 5.11a).

Location in the noisy city centre required the theatre spaces to be constructed as a "box within a box". The walls provide a sound reduction of at least 45 db and the concrete roof system, with a minimum mass of 2.5 kN/m^2 and supported on

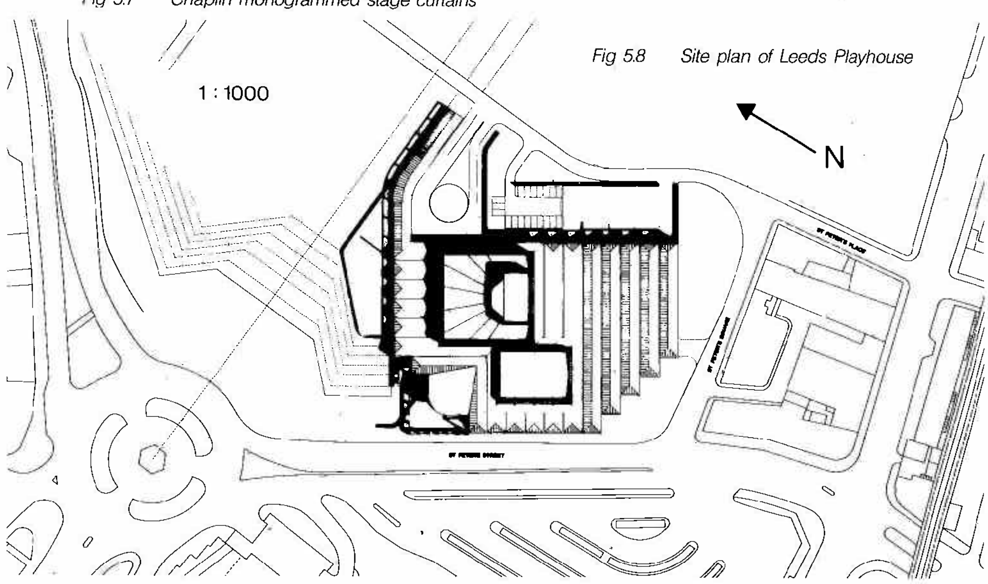


Fig 5.8 Site plan of Leeds Playhouse

Wycombe Arts Centre

RIBA Competition (first place)	
Architect	Derek Walker Associates
Theatre Consultant	John Bury
Structural & Services Engineers	Buro Happold
Quantity Surveyors	BHQS
Date	1987
Estimated Cost	£7.2m

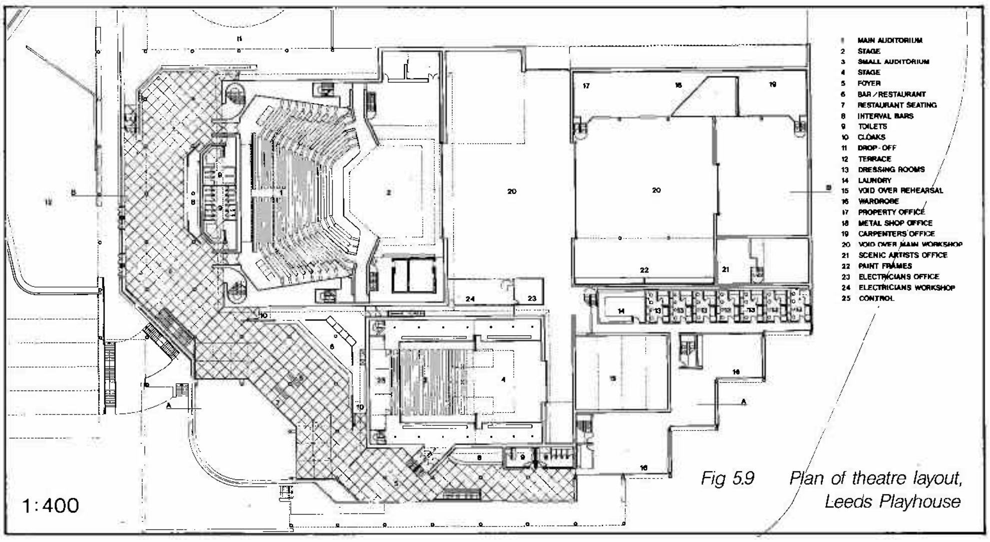


Fig 5.9 Plan of theatre layout, Leeds Playhouse

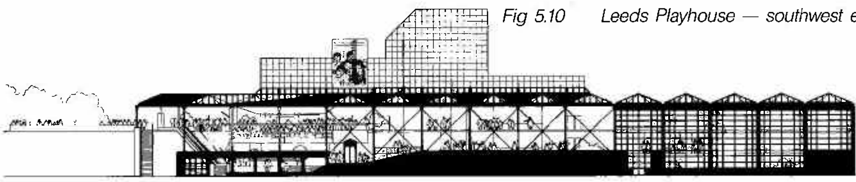


Fig 5.10 Leeds Playhouse — southwest elevation at night

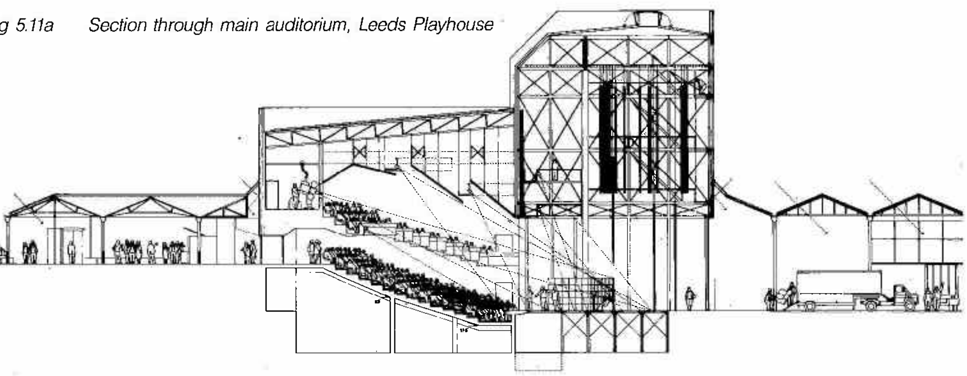


Fig 5.11a Section through main auditorium, Leeds Playhouse

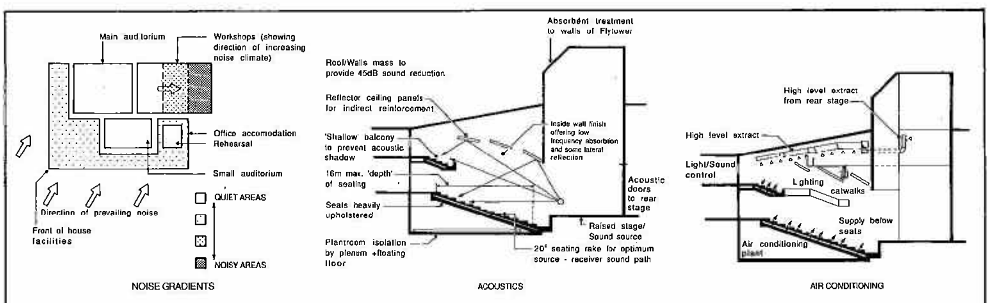


Fig 5.11b Acoustics and air conditioning, Leeds

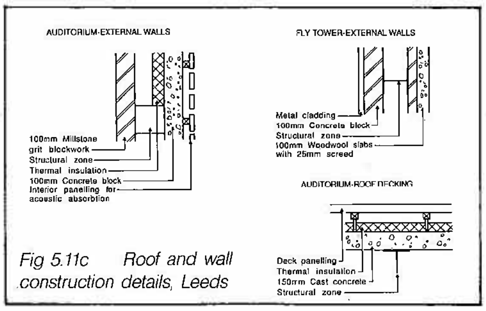


Fig 5.11c Roof and wall construction details, Leeds

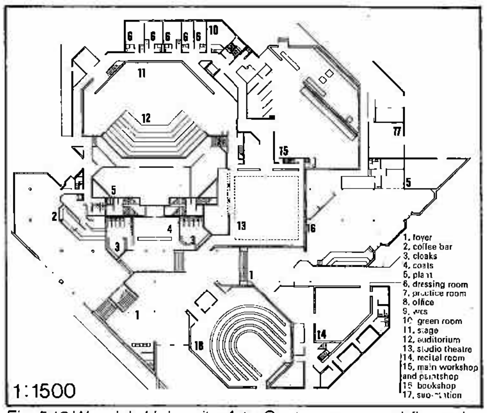


Fig 5.12 Warwick University Arts Centre — ground floor plan

deep steel roof trusses, is designed to achieve the 20–25 db noise criteria in the theatre with a reverberation time of 1–1.2m seconds (Fig 5.11b, c).

Each of the auditoria is expressed architecturally as an elegant steel panel or falence tile solid, free of penetration. For speed and ease of construction, the "box within a box" was supported on in-situ concrete raking beams and slabs, on which stood a structural steel superstructure supporting solid walls. This allowed flexible installation of the air ducting to the auditorium which was conditioned to 21°C at 50%–60% relative humidity. For economy, air was supplied from a plenum beneath the seating and extracted directly above the lighting bridges through the roof void, thus ensuring a comfortable flow of tempered air over the audience at all times.

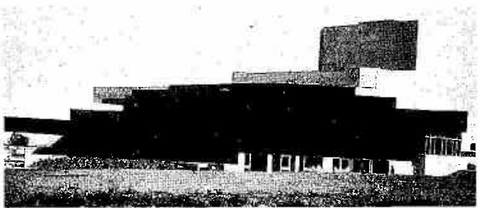
Similar principles were used for the 360-seat all-purpose studio theatre where 7 lighting bridges are supported off the roof trusses to give a total flexible space equally capable of use as theatre, exhibition or banqueting area.

Recurring Design Principles

In many respects, the conception and planning envisaged at Leeds owes much to the design in the early 1970s of theatres like Warwick University

References

5.1 Leacroft, R and H, 1984 – "Theatre and Playhouse".
5.2 Buro Happold (Internal Report) – "Theatre Royal, Bath – Review of existing heating/ventilating serving the auditorium".
5.3 Thornley D L, 1969 – "Auditoria – A review of present day HVAC Practice" Journal of Instit. of Heating and Ventilating Engineers, 37 170–185.
5.4 Croome D J and Roberts B M, 1981 Chapter 11 "Air Conditioning and ventilation of buildings" Vol 1.
5.5 Ham L R, 1987 – "Theatres – planning guidance for design and adaptation".
5.6 Architectural Review, May 1975 – "Arts Centre and Administration Building, Univ of Warwick" Vol CLVII No 939 p259–272.



5.13 *Elevation of fly tower and Arts Centre, Warwick University*

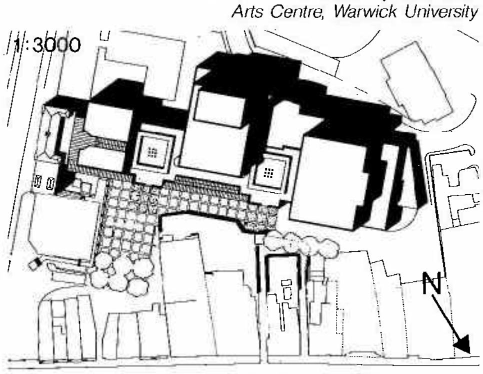


Fig 5.14 High Wycombe Arts Centre — site plan

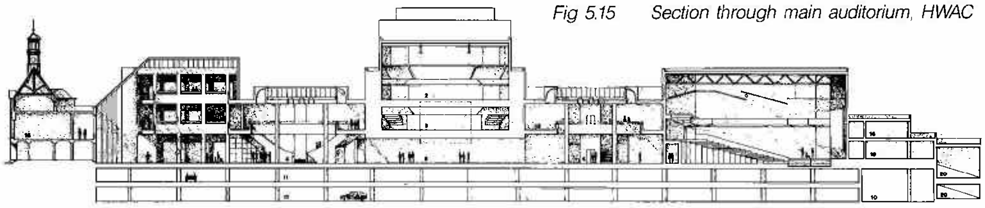


Fig 5.15 Section through main auditorium, HWAC

Table 5.3 **Leeds Playhouse Cost Plan – 1st Quarter 1985**

	Floor Area (m^2)
Auditorium 1	1,260
Auditorium 2	710
Front of House	2,610
(including plant room areas)	2,600
Workshops	2,220
Gross Floor Area	9,400

Element	Total Cost of Element (£)	Cost per m^2 including preliminaries
Substructure	590,000	68.86
Superstructure	2,050,000	239.26
Internal Finishes	280,000	32.68
Fittings	480,000	56.02
Services	1,700,000	198.40
Sub Total	£5,100,000	£595.22
External Works (Drainage only)	50,000	5.84
Preliminaries	500,000	–
Contingencies	250,000	26 60
Total Gross Cost	£5,900,000	£627.66

Inclusive of fixed furniture, fittings, stage equipment and lighting.

Table 5.2 **Wycombe Arts Centre – Cost Plan**

Section	Area (m^2)	Cost (£)	Cost per m^2
Administration	460	249,733	543
Public areas	2362	1,682,971	713
Multi-purpose hall	1709	1,794,543	1,050
Theatre	2534	2,152,489	849
Studios, Workshops, etc	1094	623,654	570
External covered walkway	232	58,000	250
First floor open terraces	200	50,000	250
Furniture and fittings		190,000	
Stage equipment		Allocated	
External areas		200,000	
Work to existing Town Hall facade		17,000	
Other alterations to Town Hall		180,000	
External services and drainage		100,000	
Total	8591	£7,298,389	£850

Arts Centre (Architects: Renton Howard Wood Levin, Engineers: Ove Arup Partnership) where John Bury was also theatre consultant, and upon which one of the authors worked. This is a centre which contains a 600-seat single space theatre, 250-seat studio theatre, and music centre together with foyers. All are designed as an intimate series of facilities together with workshops and dressing rooms suitable for both university and touring productions (Fig 5.12) (Ref 5.6). The complex, too, was constructed in fairfaced blockwork supported on structural steel frames and at the time of completion in November 1974, cost £900,260 — an average of £145.50/m^2 for the 6,143m^2 of space (Fig 5.13).

More recently, the Derek Walker Associates/John Bury/Buro Happold team have won the RIBA Competition for the new Arts Centre at High Wycombe. The same principles of theatre design and construction were utilised to provide direct access from the town centre into the facility without acoustic interference (Fig 5.14). The elevations are designed for construction in solid brickwork with interspersed stone courses. In the competition design, the 400-seat theatre was placed on two levels — again to minimise direct speech path — and has its own bar and foyer at first floor level (Fig 5.15). The 1000-seat multi-purpose hall, suitable for both musical performance of the highest level as well as live performance, exhibition and banqueting, is located with its own foyer at ground level. The 8,591m^2 of facilities required by the competition brief were estimated to cost £7.2m (£850/m^2) (Table 5.2). Clearly this compares with a similar cost plan prepared for the Leeds facility for the first quarter of 1985 (Table 5.3).

As a result of its theatrical experience in the UK, the practice has also been engaged by overseas clients on designs for performance spaces, notably at Baltimore in the USA, Baghdad in Iraq and recently in Hong Kong. With the associated Hong Kong practice, Ho-Happold, we are retained as structural engineers for the new HK$500m Lyric theatre and concert hall on the old Kowloon Pier site which is shortly due for completion.

The design of theatres requires thought and work in many areas of expertise — clearly engineering is a diverse discipline.

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