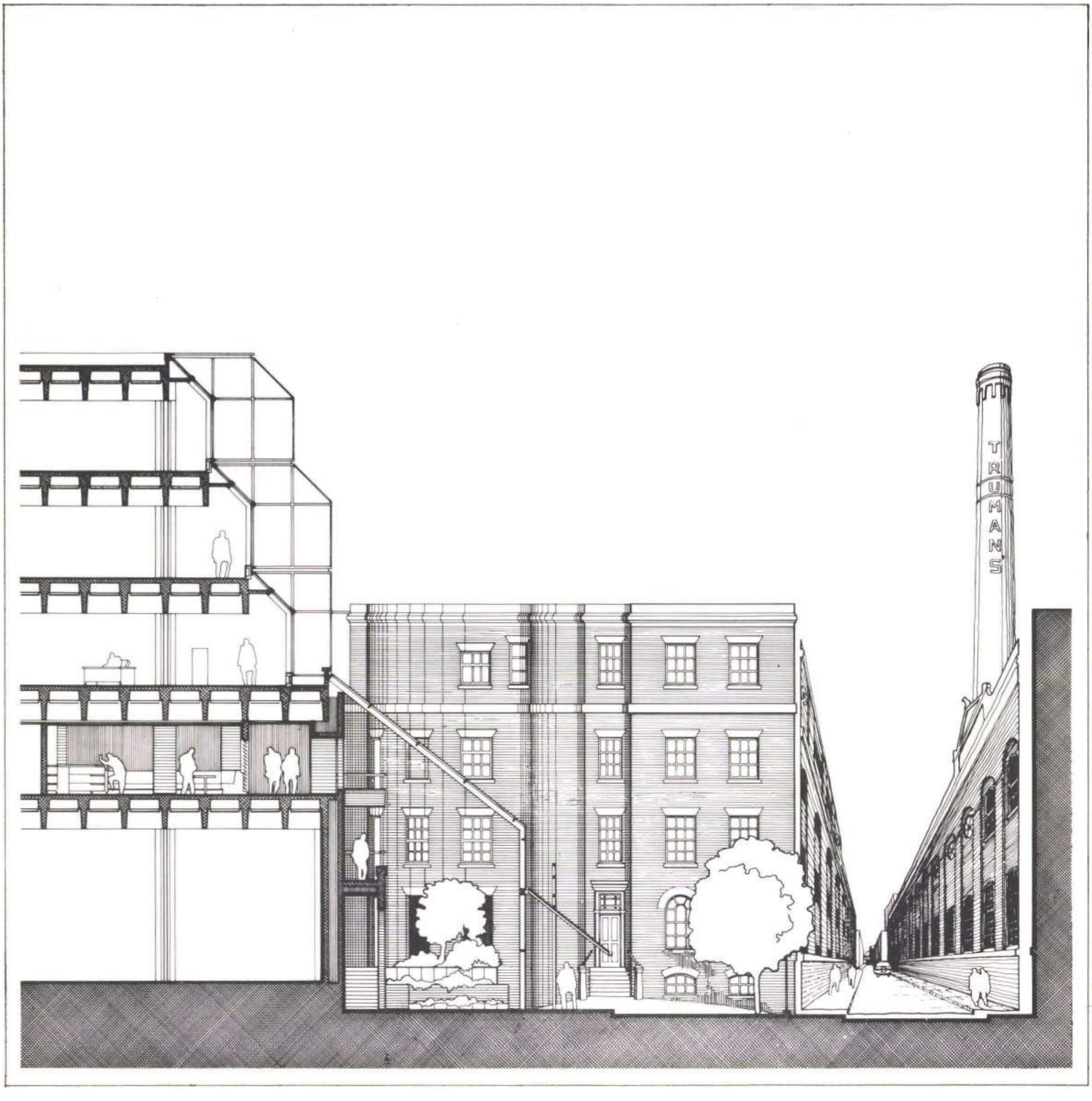
THE ARUP JOURNAL

JUNE 1977



THE ARUP JOURNAL

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Contents

Front cover: Elevation of old and new building at Truman's, Brick Lane
Back cover: Centre Pompidou (*Photo:* Poul Beckmann)

Truman, 91 Brick Lane, E1

Jonathan Gray
James Herbert

The history

Trumans the brewers have over more than three centuries enjoyed a history of building on their East End of London site. The lines of Brick Lane and its adjoining streets were in fact laid out between 1661 and 1670, and within 10 years of that time Joseph Truman, father of the more famous Sir Benjamin, was actively engaged in brewing on the site. From that small beginning, the brewery rapidly grew so that by 1840 the complex covered six acres, and the buildings which are now scheduled by the Greater London Council, and which so dominated our design ideas, were all completed.

The Directors' House had been built in 1740 by Sir Benjamin Truman as his new city residence, to be completed later with the Adam drawing room which is now the board room. The Vat House was also built in about 1740 on the east side of Brick Lane opposite the Directors' House, with the Engineers' House and the stables added to the north in 1830. The Brewers' House had also been completed before the turn of the century. It was built on the west of Brick Lane to the north of the Directors' House and separated from it by a cobbled courtyard. Records of the time describe the brewery as having 'more the appearance of a town itself than a private manufacturing establishment.' It was this essentially urban quality that we tried to exploit in our design just 200 years later.

When Arup Associates were commissioned to design the administrative offices for Truman Ltd. in March 1972, major redevelopment and reorganization had already been initiated on the site some four years earlier. Indeed, Trumans had already constructed a new

Fig. 1
Truman's brewery in 1760

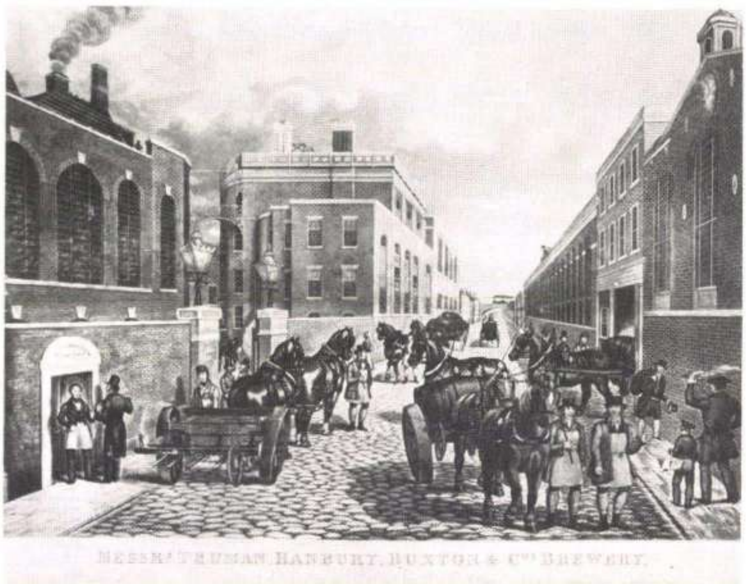


Fig. 2
Truman's brewery in 1842

Fig. 3
Aerial view of Truman's before development

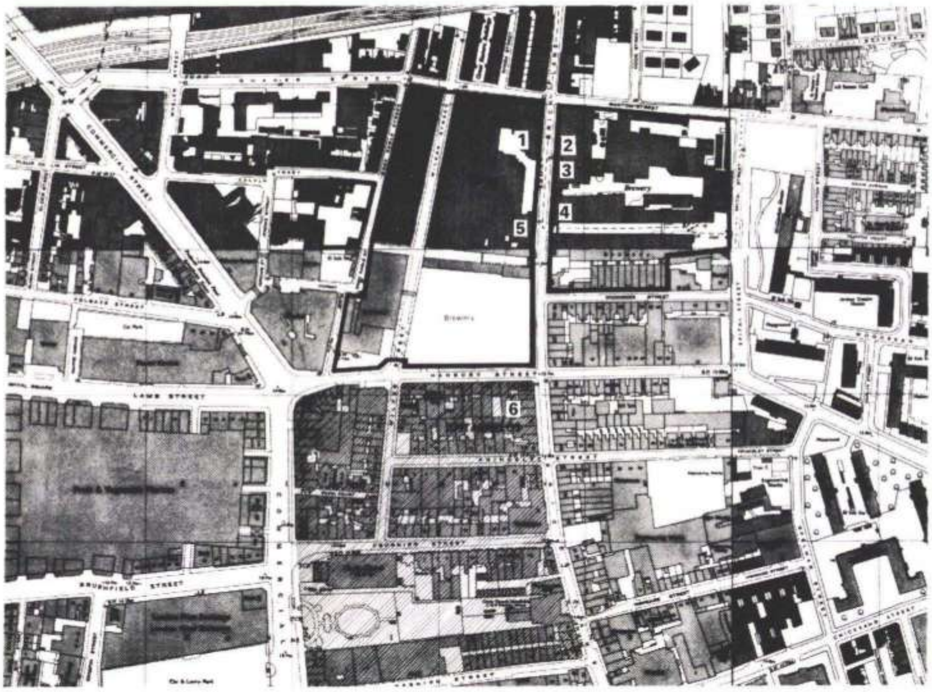


Fig. 4
Truman's site in relation to conservation area

Key
1 Brewers' House
2 Stable
3 Engineers' House
4 VAT House
5 Directors' House
6 Conservation Area

bottling plant in Hanbury Street and had been given beneficial occupancy of a new brewery building designed by a Bristol engineering firm called Gordon Smith & Partners. This firm was also just completing the design of a cold lager store, and beginning the design of a warehouse at the time we were appointed.

The area designated for the office building was a flat site of about 3,200 m² in area. It was bounded by Brick Lane to the east with the scheduled frontage of the Stable Building, the Engineers' House and the Vat House facing the site, and the Brewers' House standing on the site. To the north was Quaker Street, separating the site from some dilapidated tenement buildings which were shortly to become the subject of a compulsory purchase order. To the west was Wilkes Street which formed the major trucking access to the new brewery building to its west. To the south was the proposed new cold lager store on the west corner, and the Directors' House on the east, with Dray Walk beyond.

In May 1972 Truman Ltd. instructed a demolition contractor to clear the site of the agglomeration of stores, production, and office buildings. The Directors' House, the Brewers' House and the two spent grain silos which serve the brewery were all that was left untouched on the site. An area for a future tanker bay to the north of the cold lager store was also designated as part of the development.

Beyond the southern boundary of the site across Hanbury Street, the London Borough of Tower Hamlets had affirmed a conservation area which included Christ Church Spitalfields designed by Nicholas Hawksmoor and completed in 1727, his last London church and perhaps his finest.

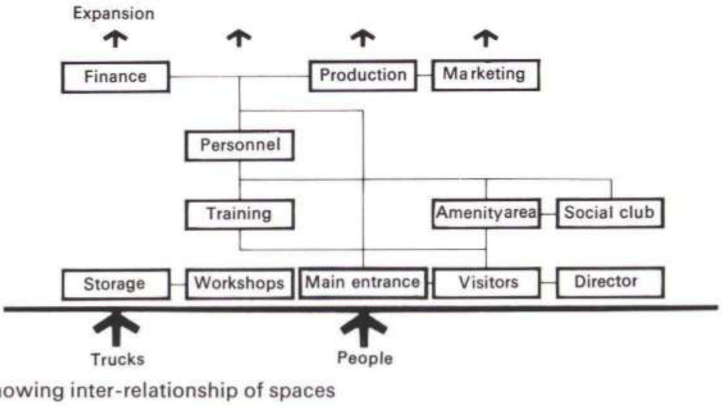


Fig. 5
Diagram showing inter-relationship of spaces

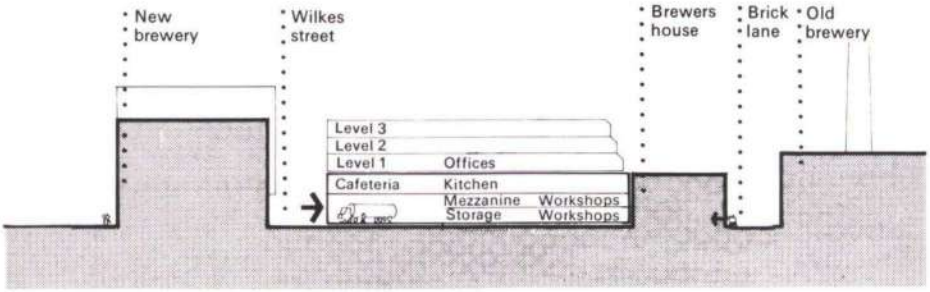


Fig. 6
Diagrammatic section

The brief

During initial discussions with Truman Ltd. the following major elements of the brief were established:

(1) An office element for 420 staff with the maximum possible room for expansion

(2) A reception area for visitors to the Brewery connected to the main entrance

(3) An amenity area including cafeteria and social facilities serving the complete site

(4) A storage and workshop area, which would not only contain all the maintenance workshops and storage for the brewery complex, but would also contain all trucking and storage facilities serving the office and amenity areas

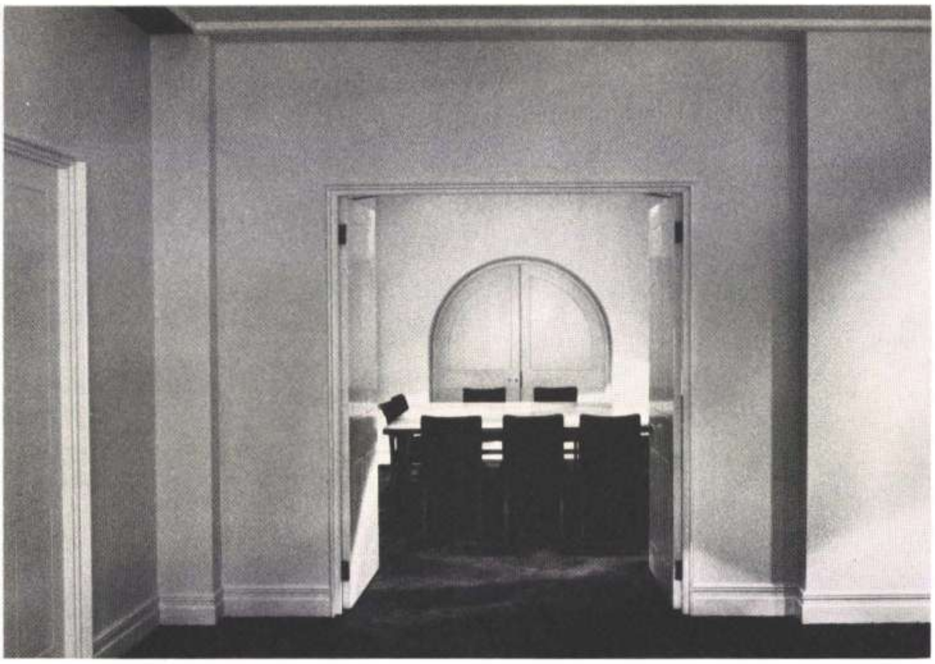
(5) The addition of cold lager tanker bays to the brief resulting from our initial proposals to design the offices over the tanker bays

(6) An investigation of the available options for treatment to the Directors' and Brewers' Houses.

The idea

The problem of the development therefore revolved around providing the maximum amount of usable floor space, whilst at the same time respecting the constraints imposed on a site close to a conservation area, and adjacent to a historic part of London.

a



d

e

b

a and **b** Conference rooms within the refurbished Brewers' House

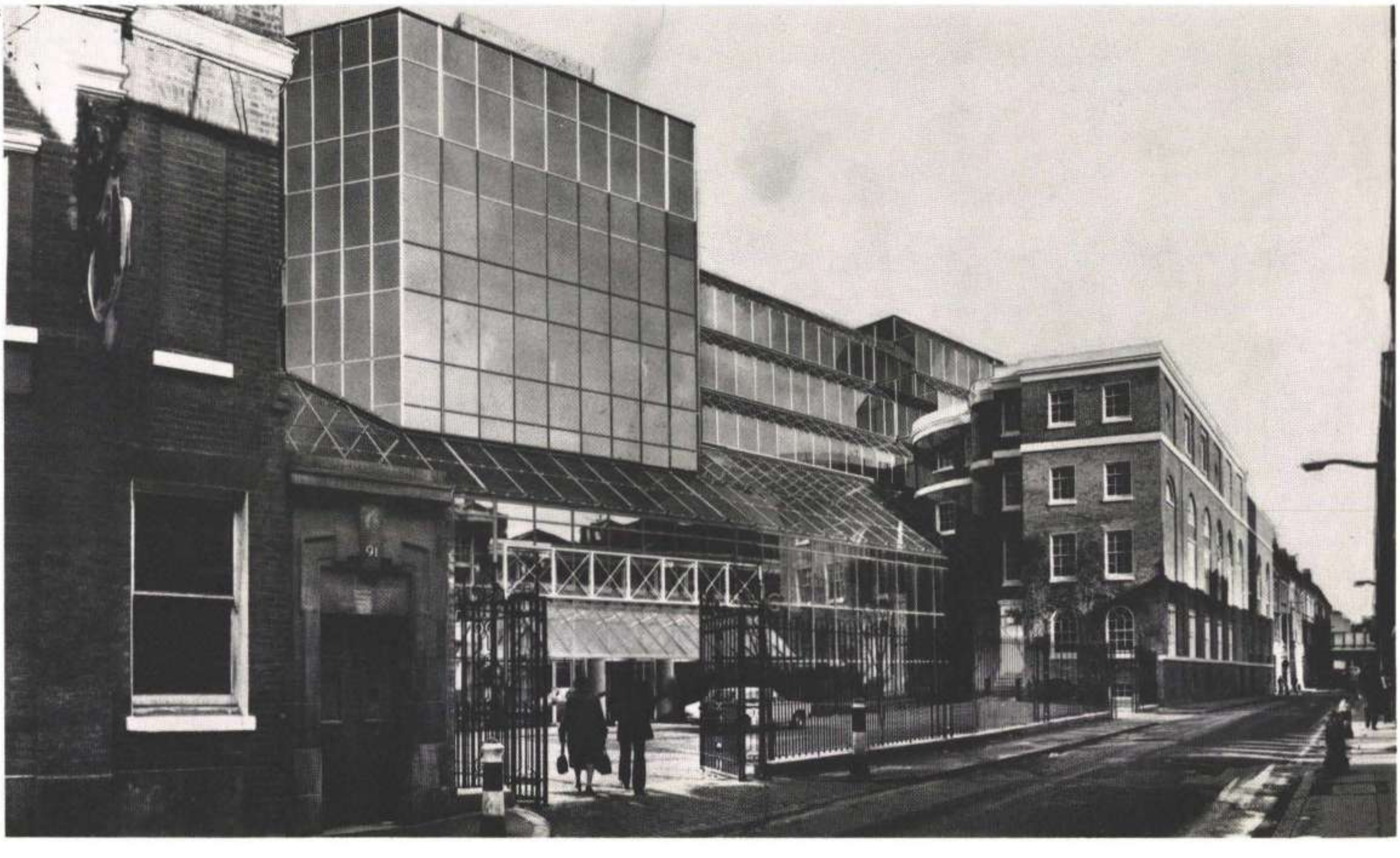
c View from Brick Lane

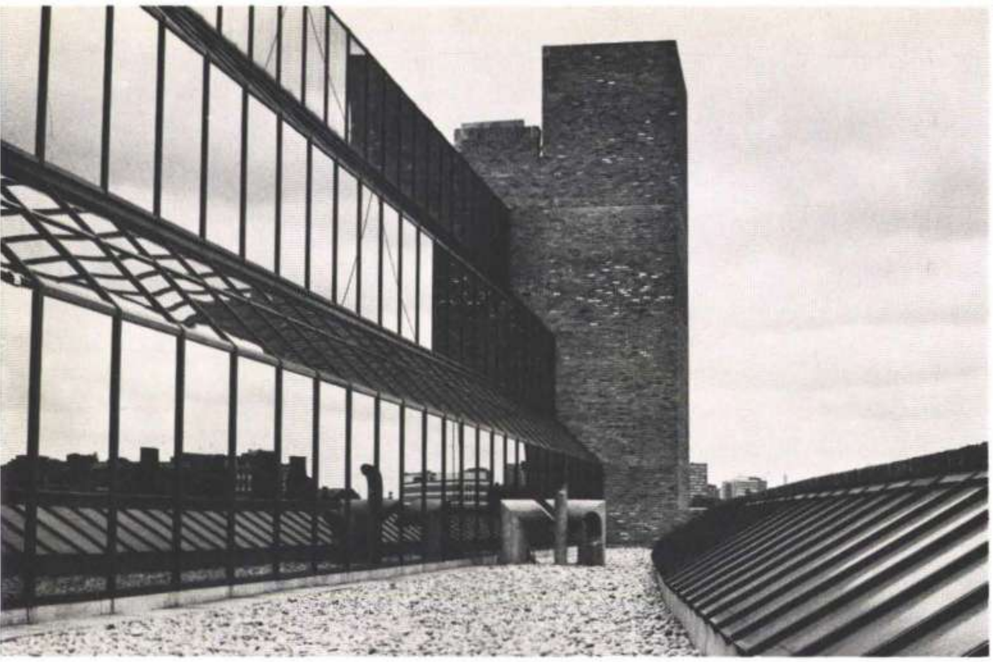
d Directors' House partially rebuilt; north face viewed through new gated railings

e to **i** Views and reflections of the new building

f

c





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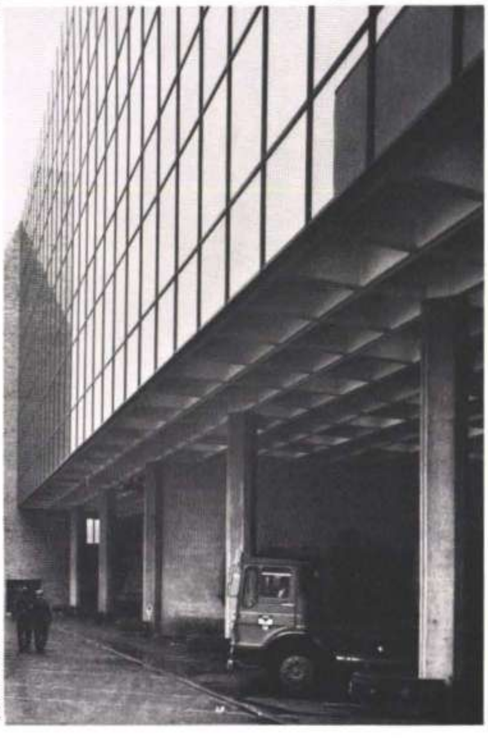


John Donat

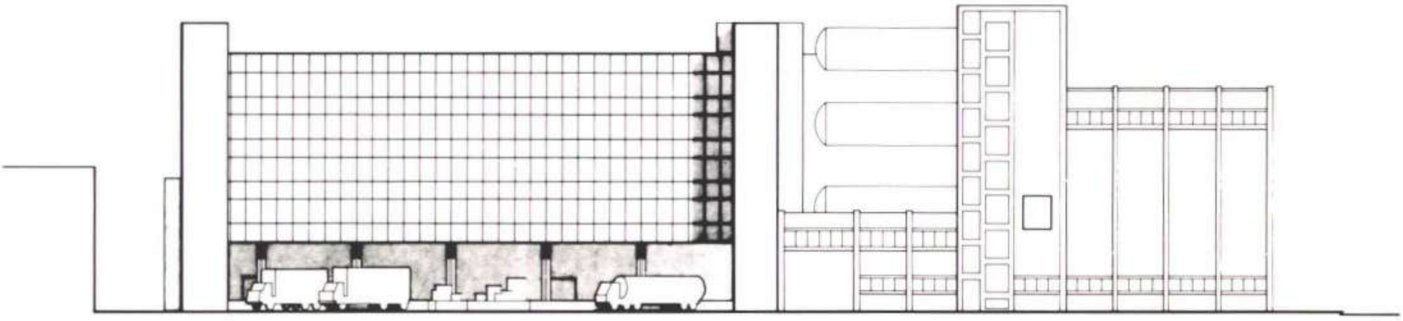


Fig. 7
Wilkes Street elevation

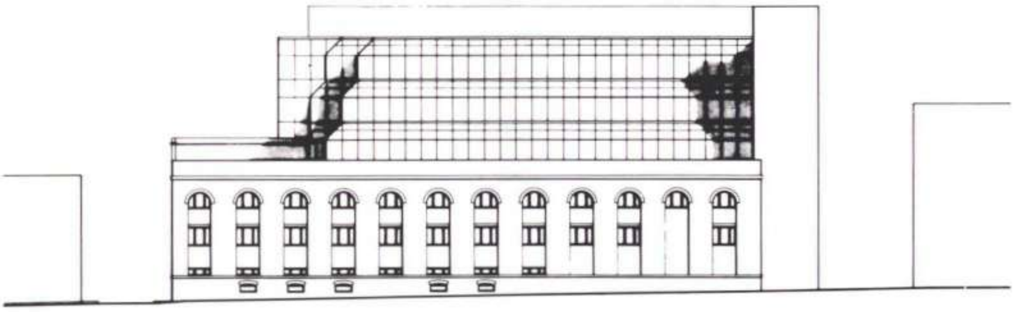


Fig. 8
Quaker Street elevation

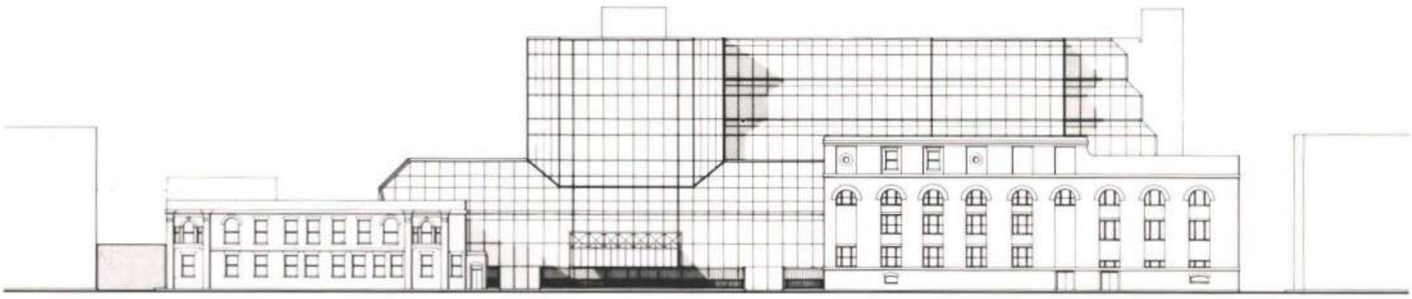


Fig. 9
Brick Lane elevation

From an analysis of the various functional requirements in the brief, it became clear that there were two types of space; office space which had a discrete activity, and the remaining space whose function related very closely to ground floor access, whether pedestrian or vehicular.

This functional separation could be augmented architecturally by expressing the two types of space as two separate buildings, one located on the top of the other; the lower one clad in London stock bricks and detailed to relate to the adjoining historic buildings, and the second building, containing the discrete office function, positioned on a newly created site at roof level to the adjoining scheduled buildings, and detailed quite differently. Such was the idea of the building. The success of this idea, however, relied on making the mass of the office building visually unobtrusive from street level, and this appeared to be in contradiction to the brief which asked for the maximum possible office space for future expansion.

The London Borough of Tower Hamlets recommended a plot ratio of 2:1 for the whole Truman complex. Examination of the total proposed brewery development suggested that a plot ratio of 2.2:1 for the office and workshop building would enable an office area to be built for expansion equal to a further 50% of the existing office needs. It would also facilitate a degree of development behind the scheduled frontages of the Stable Building at some future date and still maintain the overall plot ratio below 2:1. It was on this basis that an application for an office development permit was made to the Department of the Environment.

Exploratory discussions with the Greater London Council established certain criteria which were further to influence the design. Section 20 of the *London Building Acts* operates only within the central London Boroughs including Tower Hamlets, and applies to buildings with a particular volume of high fire risk of space or to those that are in excess of 24 m high. Owing to the storage and truck dock facilities requested in the brief, the complete Truman building was subject to the requirements of this section of the Act which specifies more stringent control over the use of materials and the methods of fire protection. During discussions with the GLC it was established that if the building were designed so that the top office floor were less than 24 m from street level, and there were adequate fire separation between areas of high and low fire risk, then the GLC would consider waiving the requirements of Section 20 for what amounted to the office and amenity areas. Thus if the building were kept within the prescribed height, this would free money within the cost plan to improve standards elsewhere.

The other bye-law that influenced the design idea was one which stated that each and every habitable space, including offices within a building designed in central London, should have a window area not less than 1/10th of the floor area of that space.

It followed therefore that in order to benefit from the GLC's offer of a waiver under Section 20, the building should logically be designed to be less than 24 m in height. Due to the available site area this would result in three levels of offices each of about 2000 m². The perimeter would therefore need to have a window area on each floor in excess of 200 m² of glass.

The further implication was that the office would need to be essentially open plan, because closed cell offices would rapidly use up the allowance of perimeter glazing required for the whole floor.

Since the conclusions from these discussions appeared to augment the ideas of what was needed architecturally for the site, it was decided that they would form part of the outline proposals. The idea of a brick podium designed to relate to the historic architecture of the street, and forming on its roof a new site for a glass clad office building was thus conceived. This solution was accepted by Trumans. It enabled the complete production department to be planned on the lowest office level and the complete sales department to be planned on the middle level. The top level could be planned for expansion.

The organization

In order to strengthen the design idea it was considered essential to rationalize the circulation and access to the building.

On the west of the site was Wilkes Street which had been closed to the public and had been planned for the main vehicular access to the new brewery, the proposed cold lager store and tanker bays. It was decided to capitalize on this situation, and formalize all vehicular access to the office and workshop building from the west off Wilkes Street, and all pedestrian and visitor access would be gained from the east off Brick Lane. This approach minimized the amount of heavy trucks passing the historic buildings and also ensured that the visitors and the users of the building would inevitably enter from Brick Lane, and would relate to and enjoy the historic buildings on its frontage.

In open plan office design, the loss in privacy for the individual using the building must be replaced by a new sense of identity. The ideas of change implied in open plan must be balanced by a recognizable sense of permanence. It seemed appropriate to establish the circulation patterns within the building for Trumans in such a way as to exploit the unique qualities of the site, and so help to create such a situation. A new cobbled courtyard was proposed, flanked by the scheduled buildings on three sides and a new brick wall on the fourth. This wall would connect the Directors' House to the Brewers' House and the circulation for the new building at every level would relate to it, and therefore to the new courtyard outside. The circulation at the three lowest levels was designed within the thickness of this wall and stepped back so that each level looked down on the one below, the whole reflecting the stepped office building above. The main entrance was conceived as an extension of this circulation idea being a glazed conservatory and designed as part of the external courtyard using walls of London stocks and floors of Yorkstone.

The amenity floor was positioned at the highest level of the brick podium with the principal circulation route of that level overlooking the main entrance. The remaining spaces were planned around a central kitchen. London stock bricks were again used extensively to emphasize the essentially external quality of these spaces, as well as to create a sense of permanence. Likewise on the three office floors circulation was planned around the perimeter of the floor exploiting the views of the historic surroundings. All the service spaces, including a coffee lounge on each floor, were grouped on the south end and built in stock bricks.

It was also decided to house the reception area for the visitors to the brewery in the refurbished Brewers' House. The southern end of the building containing the domestic rooms had been used as offices, and the northern end had been the experimental brewery, containing a four storey void full of brewing plant and equipment.

The proposals were to construct floors in the experimental brewery at the same level as those in the southern end of the building. This gave sufficient space to house not only the visitors' reception area, but also a number of conference rooms, the training facilities, and the personnel department.

Planning all these spaces which required closed offices in the Brewers' House enabled the number of partitions in the open plan office of the new building to be minimized, and thus strengthen the concept of open plan. This principle was also used when we were given the brief in August 1972 to replan the scheduled Directors' House.

We proposed that the Directors' offices were retained on the first floor related to the board room, and the remainder of the building was planned to contain closed offices, the medical suite, the cashier's department, and other small scale activities.

It is worth mentioning that we renovated all the interiors of the Directors' House, except the Adam board room and its associated corridor, which sadly were omitted from our brief.

Our proposals for both the Directors' House and Brewers' House were discussed with the GLC Historic Building Division, on whose consent planning permission was dependent.

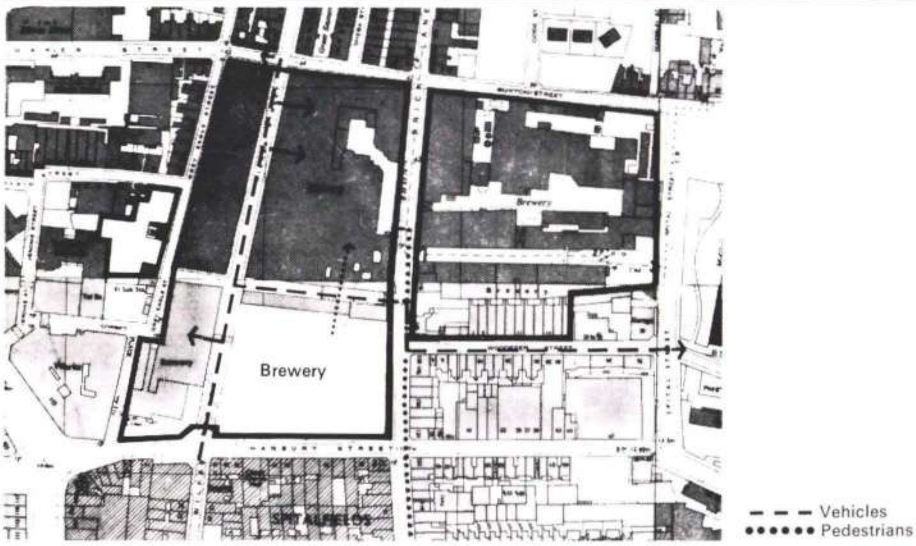


Fig. 10
Site circulation

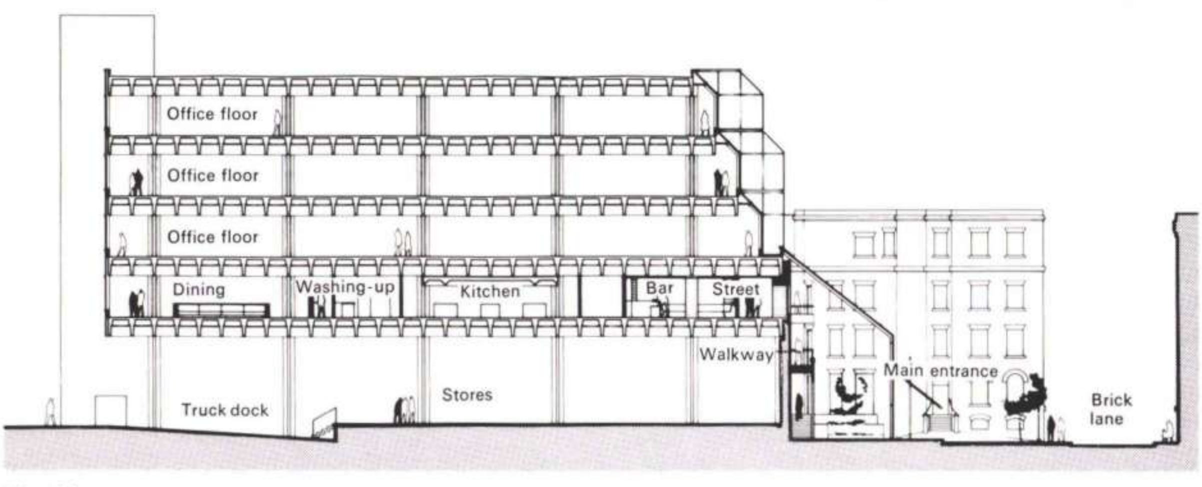
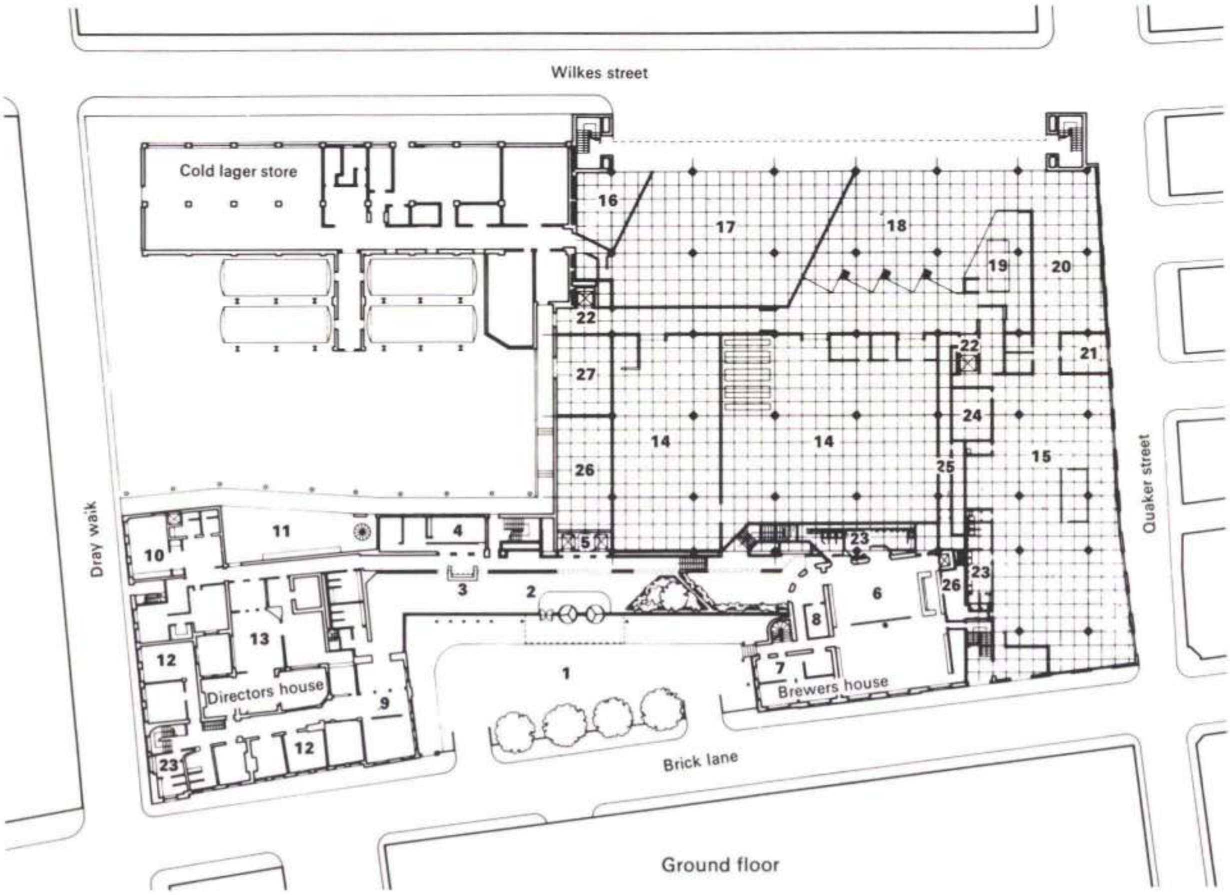


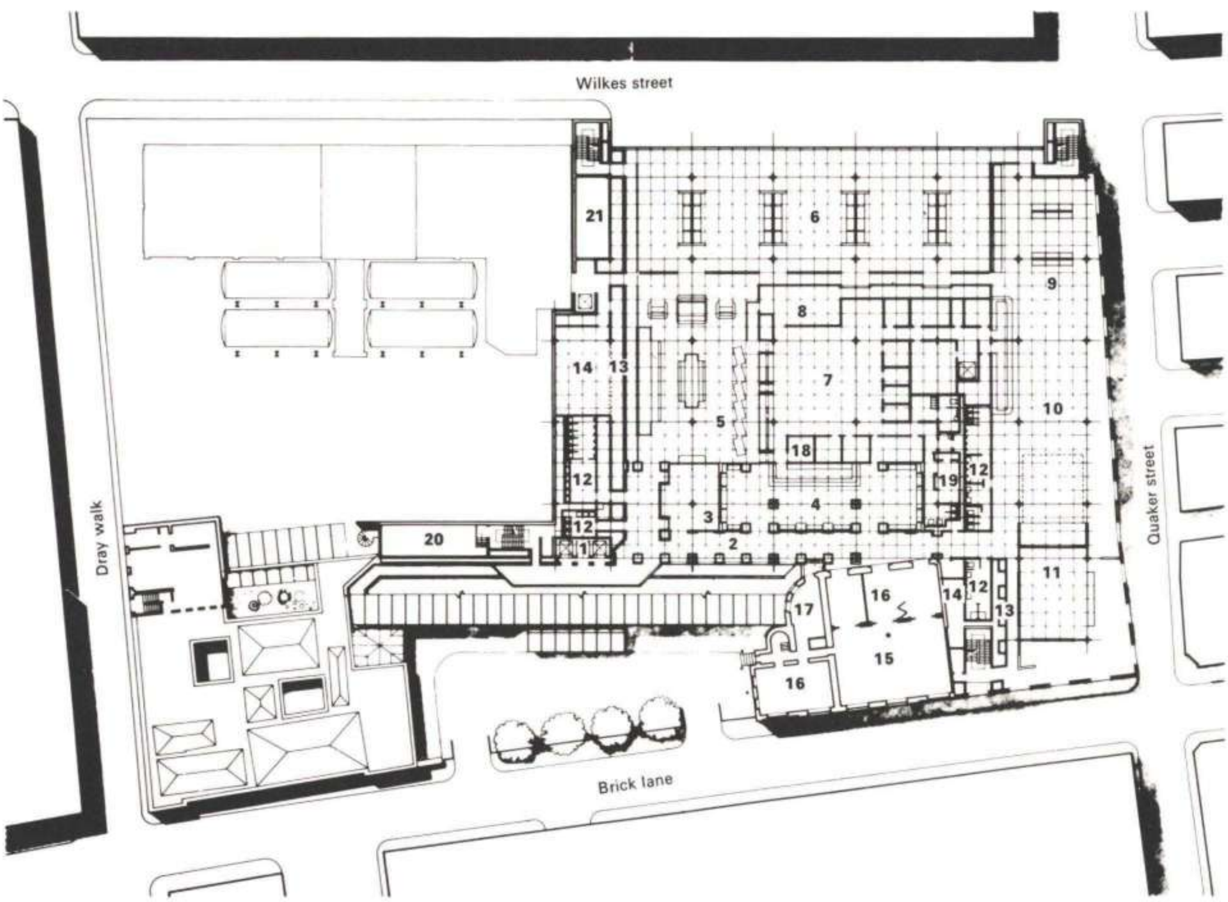
Fig. 11
East-west section



Key

1 Courtyard
2 Entrance hall
3 Reception
4 Security
5 Lifts
6 Brewers house main reception room
7 Security offices
8 Coats
9 Directors house entrance hall
10 Medical suite
11 Rear entrance and cycle yard
12 Offices
13 Wages dept.
14 Stores
15 Millwright and coppersmith
16 Silo
17 Tanker bay
18 Truck dock
19 Compactor
20 Scrap metal/stores
21 Acid store
22 Service lift to office floors
23 Toilets
24 Lockers
25 Service corridor for Brewers house
26 Plant
27 Transformer

Fig. 12
Ground floor plan



Key

1 Lifts
2 Street
3 Shop
4 Lounge bar
5 Servery
6 Dining
7 Kitchen
8 Wash-up
9 Coffee lounge
10 Social club
11 Games room
12 Toilets
13 Airwall
14 Plant space
15 Trade training centre
16 Meeting/ conference rooms
17 Ante-room
18 Kitchen offices
19 Kitchen changing rooms
20 PABX equipment room
21 Silos

Fig. 13
Amenities floor plan

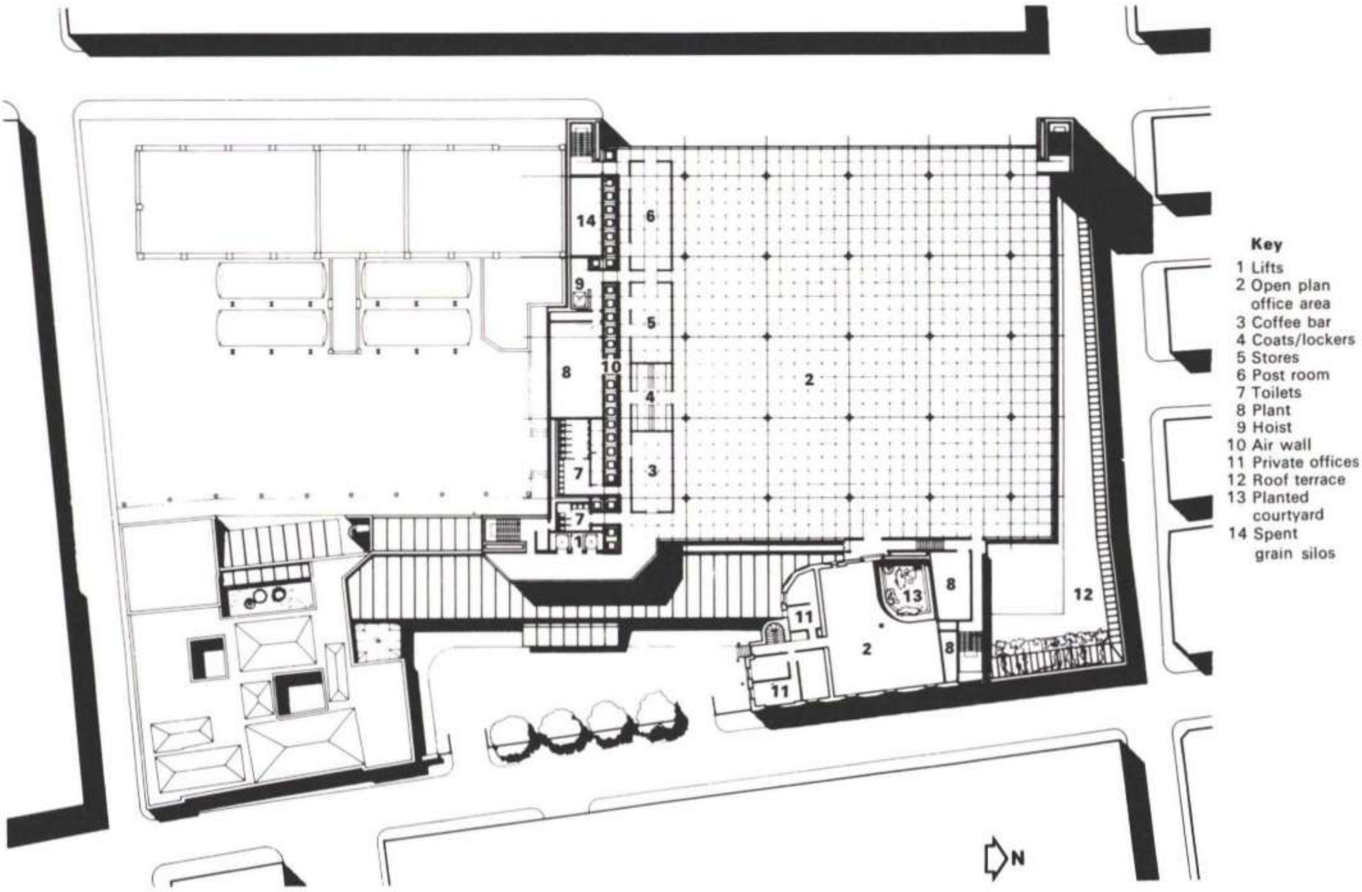


Fig. 14
Typical office floor plan

In parallel with this, an examination was being made of environmental servicing and the location of the related plant. In an office building for Penguin Ltd., Arup Associates had designed a coffer ceiling with a second precast concrete slab on sleeper walls constructed above to form a return air plenum. It was decided at an early stage in the design to adapt this idea to the different requirements of Truman Ltd. These included the facilities for closed offices, in any location on an office floor, and also the option of fixing future services within the air void.

The system that was designed for Trumans consisted of a supply and extract formed from builders' work ducts. This was achieved within the design of a 1.2 m deep, 1.5 m² coffer in which a 300 mm deep air way was cast in one direction only just below the slab, and at the top of the ribs. An airtight ceiling panel was fixed below these air ways, and each row was connected to the supply and extract system alternately. By connecting rows of supply or extract diffusers in every alternate coffer, a system was developed that enabled any size of office to be planned in any part of the office floor with full environmental servicing. Complete separation between the supply and extract system was also achieved by means of the concrete construction. A limited number of additional services would be fitted into the air voids as and when required. A light fitting set high in each coffer gave the lighting and spatial effect which had been exploited at Penguin Books office building.

A variable volume air conditioning system was used in conjunction with this coffer designed to give winter conditions of 21°C at 40% relative humidity and 23.5°C at 55% relative humidity in summer. The motorized diffusers which only supplied air at below ambient room temperature were activated by thermostats mounted behind the open egg crate diffuser of the light fitting. These thermostats controlled up to six diffusers and avoided the necessity of having protuberances on the columns.

The success of an office space is to a large extent dependent on the design of its ceiling. The Trumans design embodied the benefits of establishing a strong concrete grid over the ceiling with low brightness lights set high in the coffer, with a fully demountable ceiling. By using builders' work ducts, the air zone could be reduced to only 300 mm high which gave advantages in maintaining the building below 24 m in height. Exposing a large amount of concrete to the office space made the building more thermally inert, thus reducing the load on the air conditioning plant.

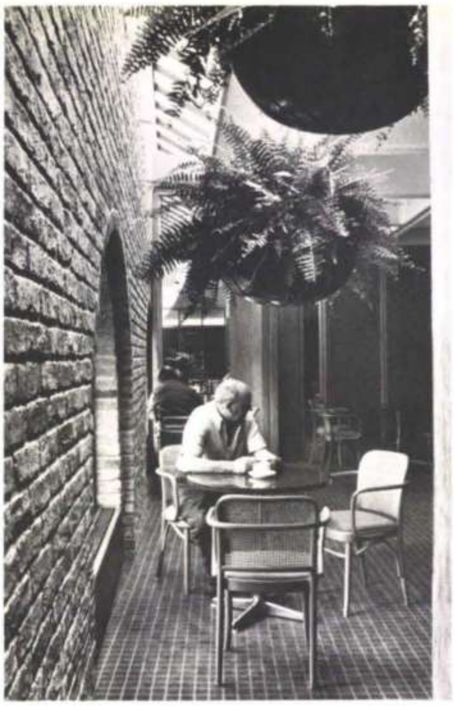
The design of the deep coffer form of the floor was based on re-usable GRP moulds, and it was the contractor who took the initiative in proposing permanent formwork in glass fibre reinforced cement. GRC, using Pilkington *Cem Fil* fibres, had been on the market for less than two years at this stage, and a series of studies was necessary to satisfy ourselves that the idea was feasible. We were grateful for the help our own R & D gave us, in suggesting a series of studies to establish the mechanical properties of the material, and quality control methods using different production techniques. Its ability to withstand loads during construction, as well as its behaviour when exposed to fire, was also examined.

The inevitable logic of having alternate bands of supply and extract air across the full width of the building on each floor level was that across one complete side of the building there should be vertical connecting ducts linking the air floors, as they were called, at each level. This became known as an air wall and was also to be constructed as a series of builders' work ducts, related to the principles of builders' work ducts established for the air floor.

On the south of the site was the area designated for the cold lager store, with a related expansion zone for the construction of future lager tanks. The Trumans' brief required that a 3 m space be left between the site designated for these tanks and the building designed by Arup Associates. The conclusions reached were therefore to build a vertical builders' work plantroom through the full height of the building separated from the office floor by the vertical air wall to reduce sound transmittance. Access doors opening out into the 3 m free zone between the buildings were provided for the plant at each level. By providing a lifting beam on the roof, equipment could be lowered from each level to the ground. The size of the plantrooms needed only to be large enough to contain the equipment. The normal additional space required to manoeuvre equipment for replacement was thus eliminated. The lifts, the lavatories, two of the four escape staircases, and the 20 m high spent grain silos were all located along the southern face of the building to form a 7.5 m band thereby servicing an unobstructed area of usable floor space immediately to its north.

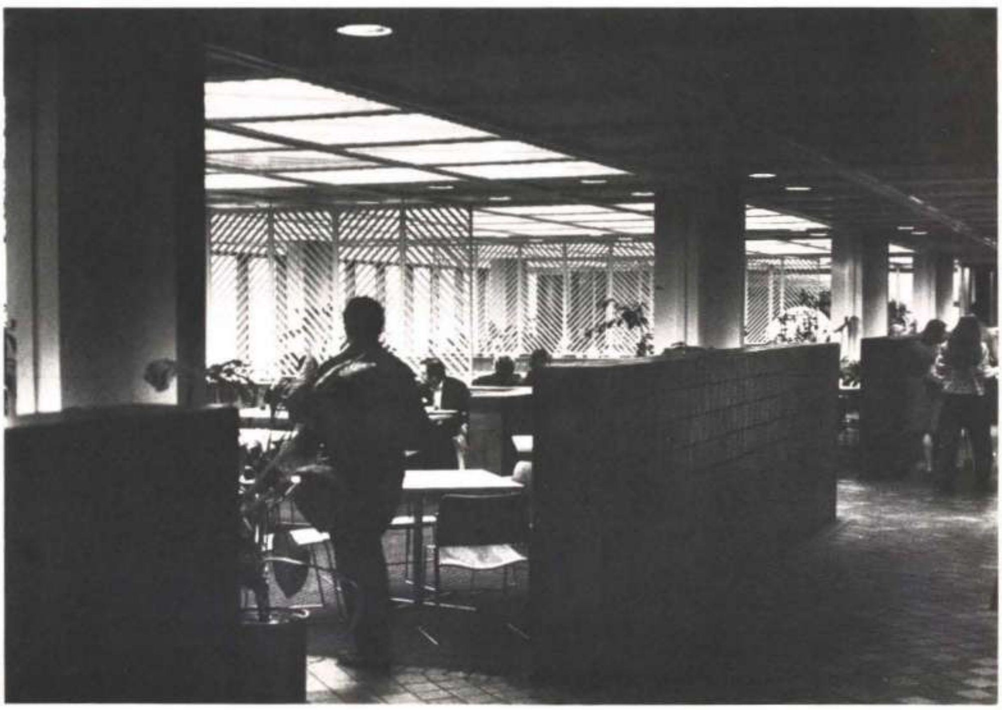
The service zone was constructed in brickwork without windows to avoid solar gain. The essential glazing for the office was therefore confined to the elevations with least solar gain.

The design of the curtain wall was also of



John Donat

John Donat



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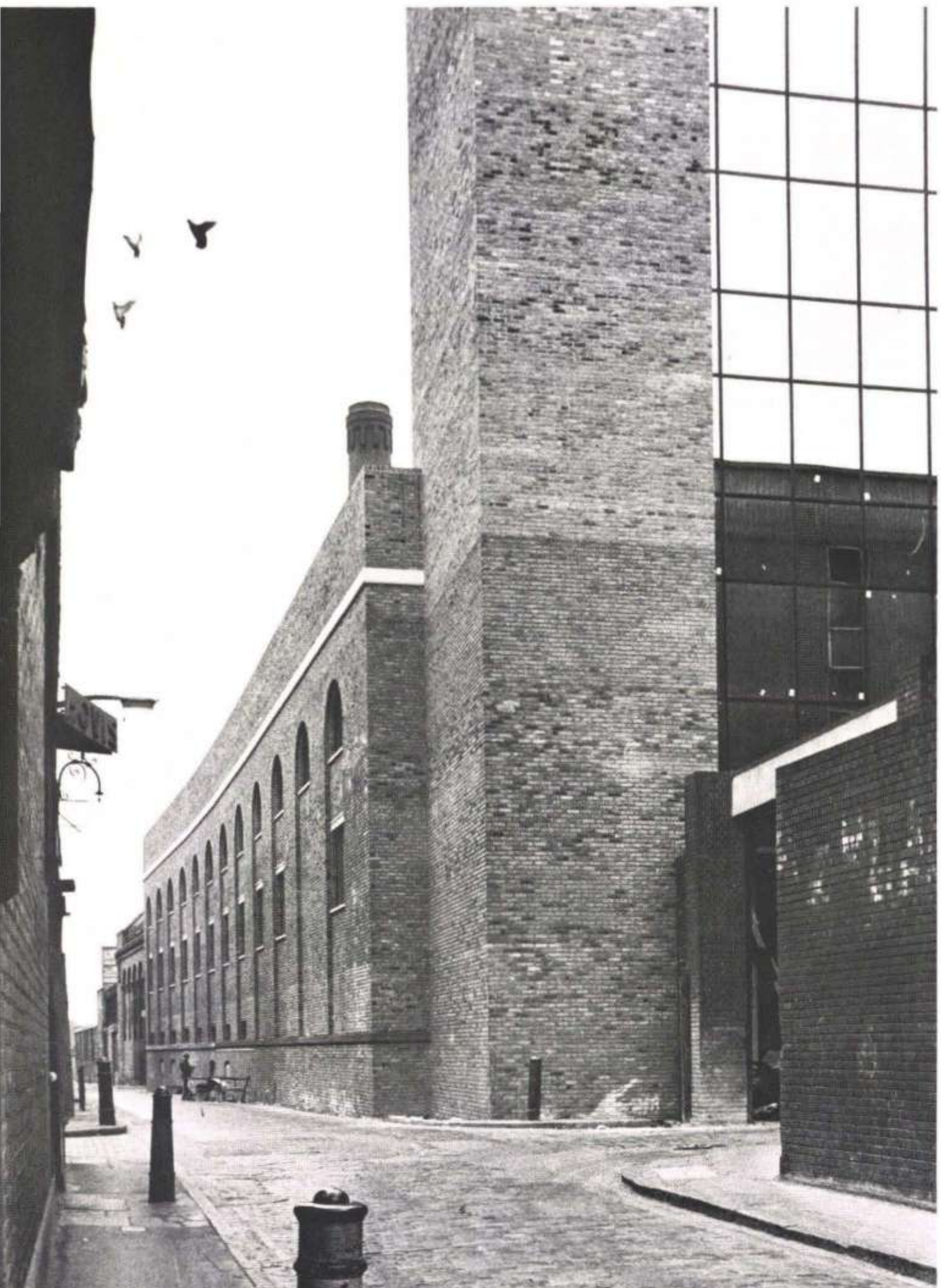
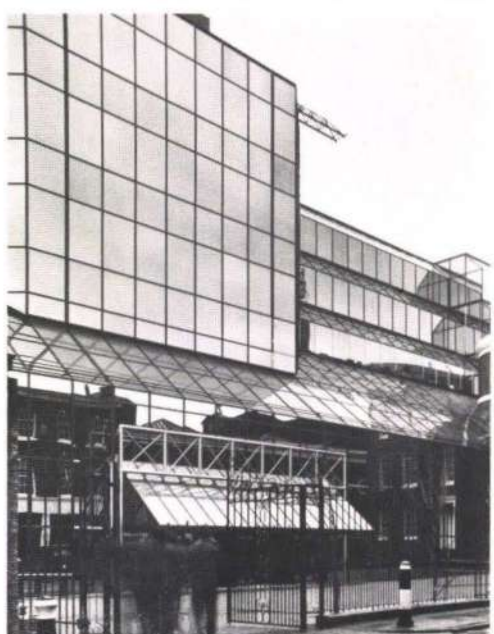




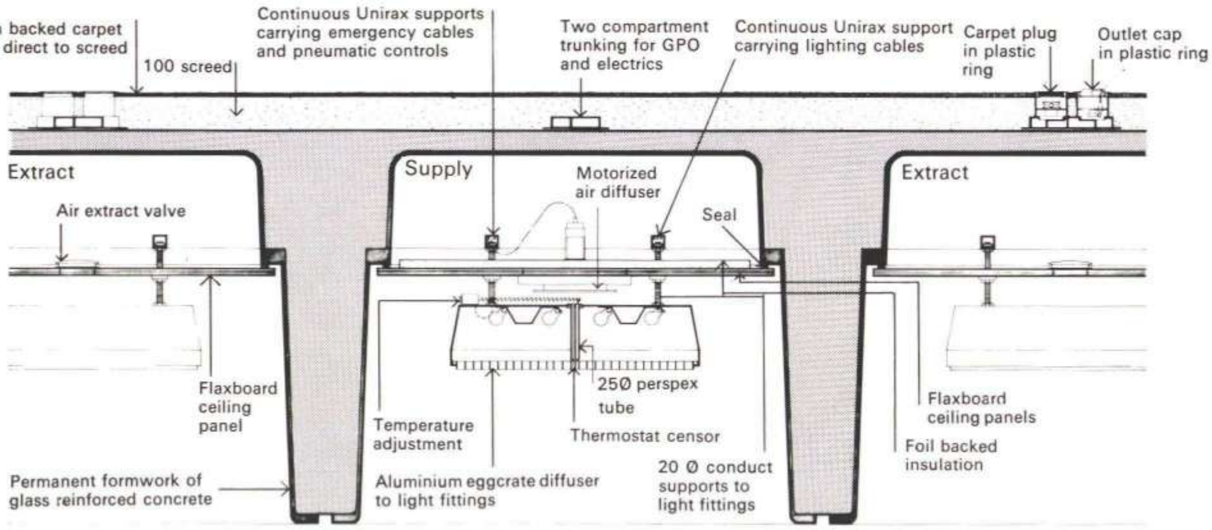


Fig. 15
Typical coffers

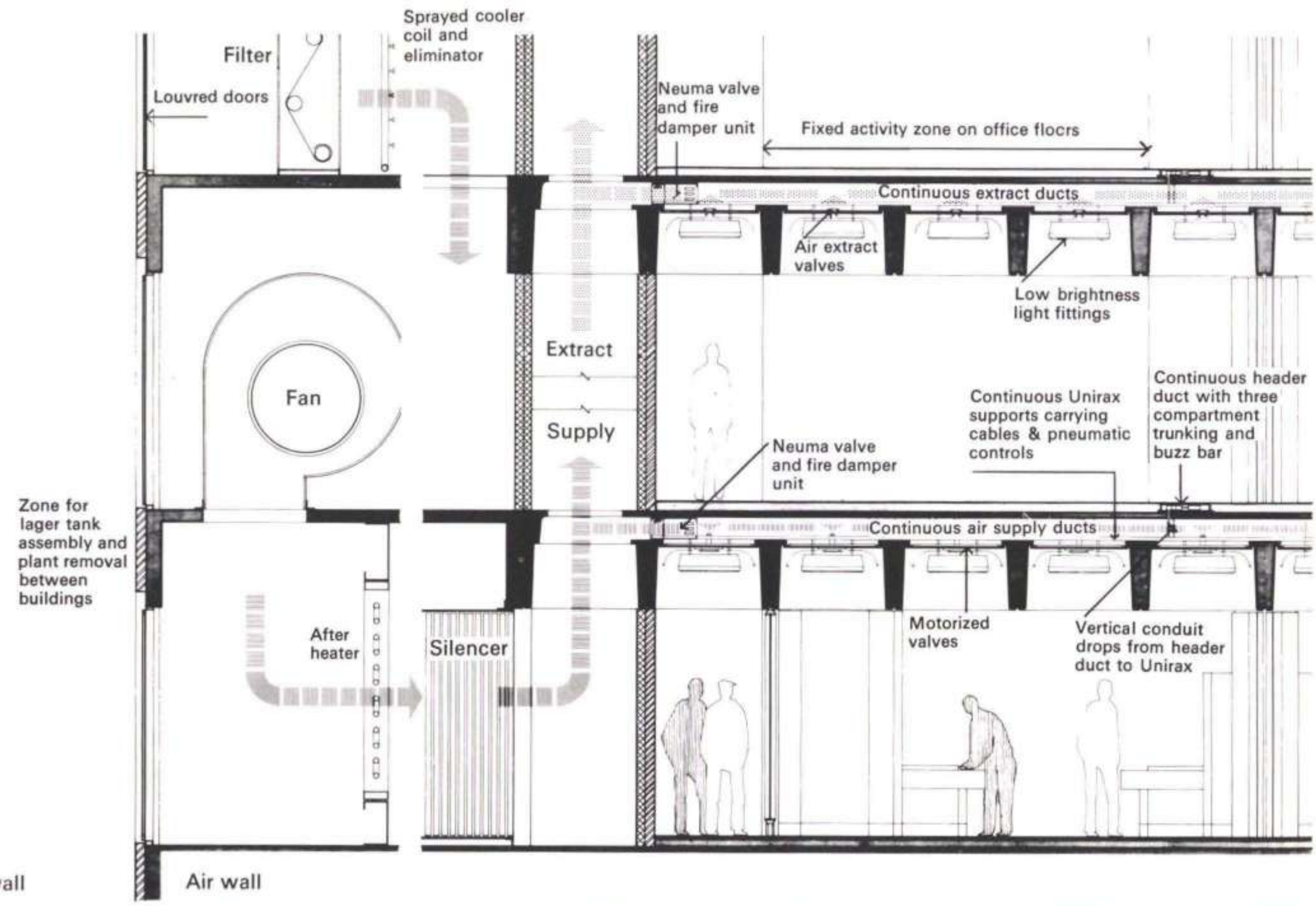


Fig. 16
The air wall

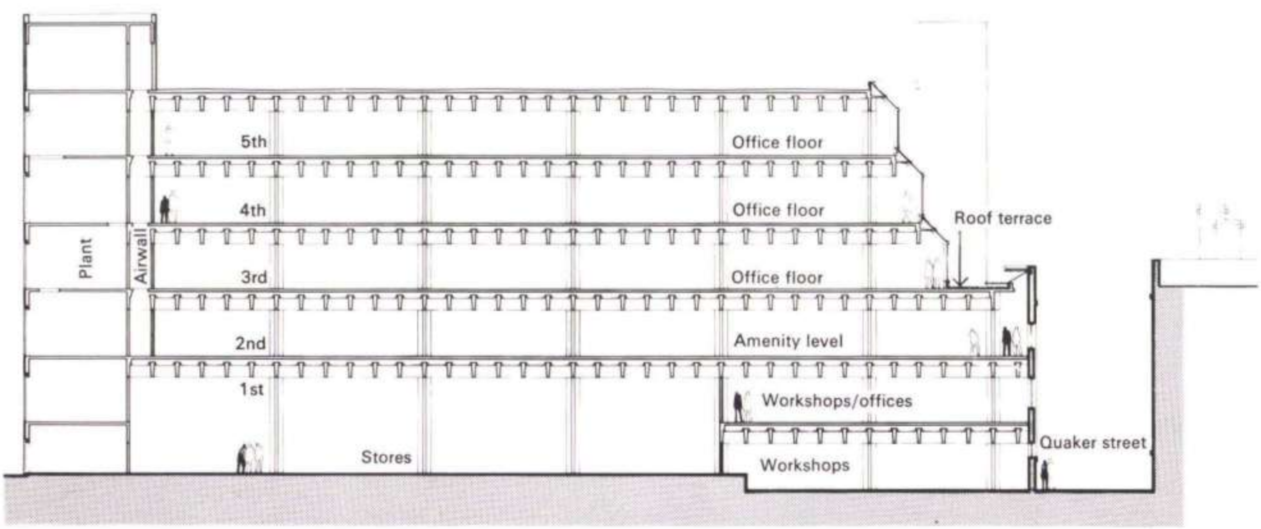


Fig. 17
North-south section

Figs. 18 & 19
Interior views of the offices

Fig. 20
Amenity area

critical importance to the air-conditioning plant. The required proportion of glazing on the three elevations amounted to 75% of the cladding to each floor. The choice of systems narrowed itself to:

(1) A double skin using dark glass and an extract plenum system

(2) A double skin with blinds between glazing or

(3) A double glazed sealed unit using reflective glass.

Of these solutions the last option using reflective glass proved to be the most interesting. The performance of certain of these glasses was such that 75% glazing in reflective glass was equivalent to just over 20% in clear glass terms of summer cooling load. The need for the use of blinds would also be minimal in summer. This was considered an added advantage in a building that was 45 m in width and enjoyed unique views over the City and the East End of London. Since the circulation was planned around the perimeter of the office floor no blinds have been needed.

The most exciting feature of the reflective glass, however, was in the effect it had on the architectural concept of the building. The office floors had been stepped back at each successive floor level with a sloping glass roof connecting each floor. By using silver mirror glass the sky would be reflected in these different sloping planes to reduce still further the visual mass of the office building and thereby strengthen the architectural idea.

The use of mirror glass also opened options for the glazing of the main entrance area, which had been planned as a link area between the Brewers' House, the Directors' House and the new building. By taking the silver mirror glass down to the ground and setting it back from Brick Lane, a new urban courtyard could be formed contained by the three scheduled frontages of the historic buildings, with the fourth side of mirror glass reflecting the Georgian Stable Building and thereby completing the square.

During the construction of the Truman building more than three quarters of a million secondhand London stocks were bought from demolition contractors all over London. It seems a paradox that the use of these bricks which was so essential to our ideas of conservation was only made possible through the massive amount of destruction that was taking place at the time the Truman building was being designed and built.

Acknowledgements

It is important to acknowledge the particularly active role that Truman Ltd. played throughout the design of the building. Before we had been appointed they established a Design Group with four representatives from different parts of their organization and it was this group that we met on a regular basis throughout the design. It is fair to say that Trumans proved to be an exacting client who insisted on formal presentations of every aspect of the design.

We were, however, given the opportunity of presenting our design at both outline proposals and scheme design stage to their Board of Directors as well as to a very large representative body of people who are now using the building. In retrospect it is satisfying to note that through this degree of consultation the brief and the design solution have become inextricably entwined.

Credits:
Client:
Truman Ltd
Architects+Engineers+Quantity Surveyors:
Arup Associates
Main contractor:
Holland Hannen & Cubitts Ltd
Photos:
Arup Associates, except where stated

Clinker Wharf Chittagong

David Brunt

Introduction

Towards the latter part of the 1960s Ove Arup and Partners were appointed as engineering design consultants for several industrial projects in what was at the time East Pakistan. As these projects reached the construction stage resident engineers were sent out to the respective sites and an Ove Arup and Partners presence in the capital, Dacca, was established.

When local representation is set up and keen interest in the local construction industry maintained, opportunities for engineering involvement usually present themselves and so it was in the spring of 1969 when we were appointed by a consortium of two local construction companies, Stoneville Engineers Ltd. and The Engineers Ltd., to provide a design and supervision service within the terms of a design and construct contract that they had been awarded by the East Pakistan Industrial Development Corporation (EPIDC).

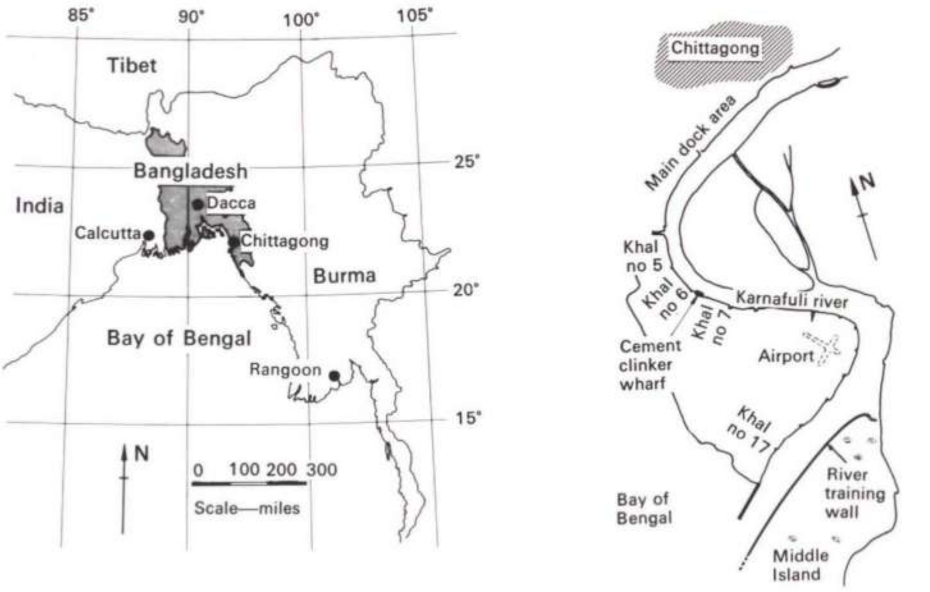
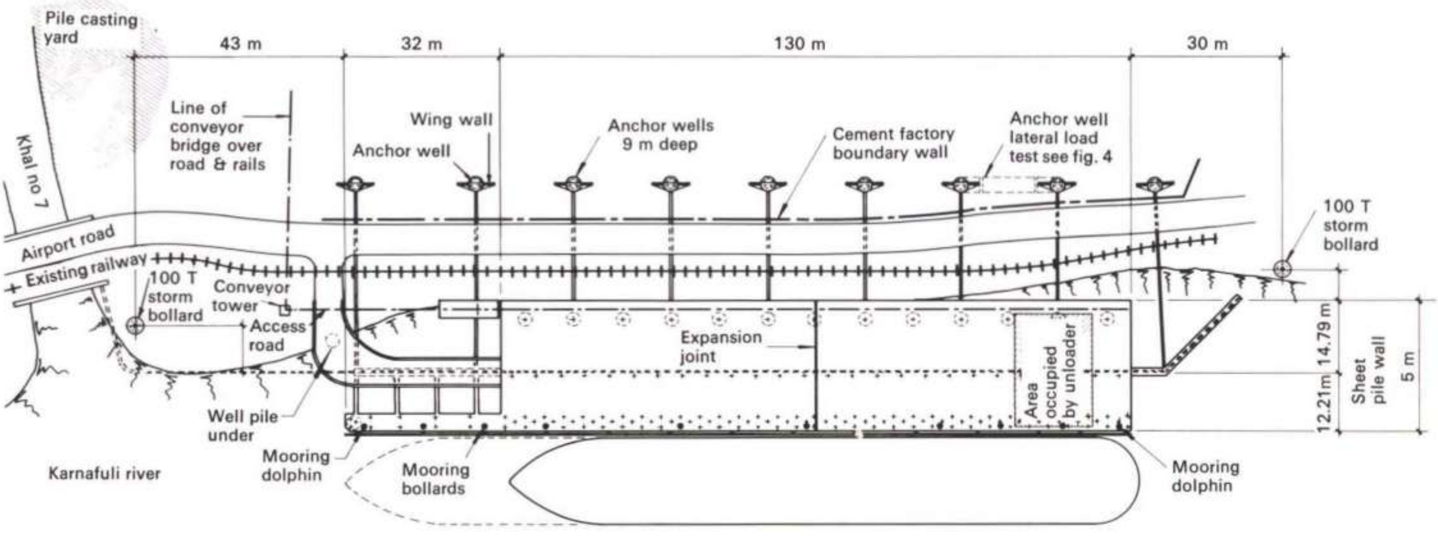


Fig. 1a
Site location Chittagong



Plan of wharf

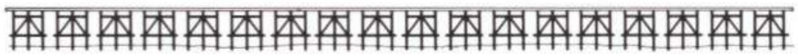


Fig. 1b
Above water elevation on fendering system

This was for the provision of a wharf on the banks of the Karnafuli river in Chittagong to take 10,000 tonne vessels bringing in clinker to feed an adjacent cement factory already partially constructed under a separate contract.

The contract for the wharf included a construction programme of 18 months total duration with a requirement that approximately one third of the wharf deck be completed within nine months so that erection of a large mechanical unloader, being provided under a separate contract by a French company, Fives Lille Cail, could be carried out simultaneously with construction work on the remainder of the wharf.

Design considerations

Chittagong itself is several miles inland from the Bay of Bengal and straddles the Karnafuli river which meanders through alluvial deposits before reaching the sea. (See Fig. 1). River training work downstream of Chittagong had been commenced several years earlier, and the contract for the clinker wharf included a requirement that a steel sheet pile river training wall be included as an integral part of the wharf.

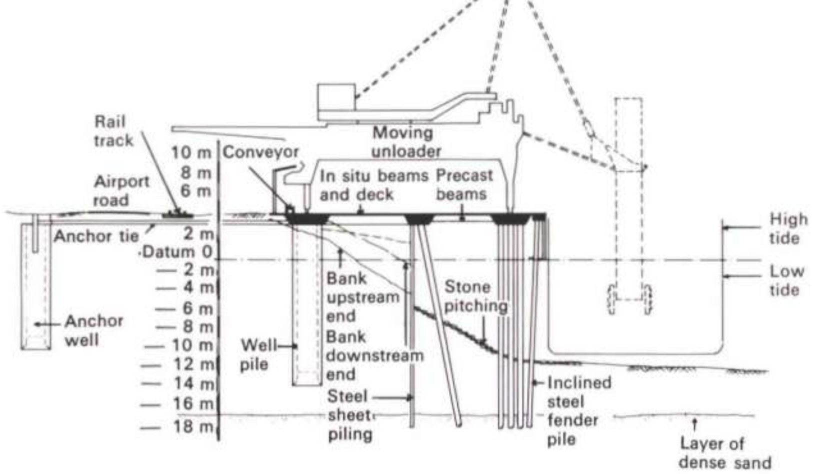
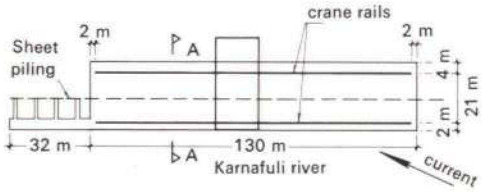
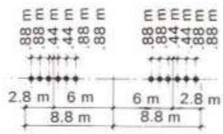


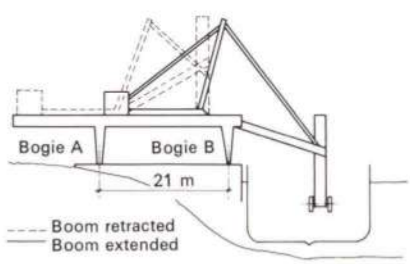
Fig. 1c
Typical section through wharf



Plan of wharf



Layout of bogies



Section A – A showing positions of unloader

Vertical loading

Live load 30 kn/m², under unloader 10 kn/m²
on substructure 15 kn/m² ,, ,, 5 kn/m²

Due to crane
These loads are influenced by
i) position of unloader boom
ii) wind force and direction

Wind blowing perpendicular to longitudinal axis of wharf

Height of top of cabin above M.S.L. ≏ 50ft
@ 45 m.p.h. p = .33 kn/m² = 7 lb/ft²
@ 90 ,, p = 1.32 kn/m² = 28 lb/ft²
@ 150 ,, p = 3.63 kn/m² = 77 lb/ft²
Note: bogie = 1 train of 6 wheels.

Bogie loads (kn)						Loading condition
45 m.p.h. wind		90 m.p.h. wind		150 m.p.h. wind		
A	B	A	B	A	B	
565	1240	565	1240	565	1240	boom extended no wind
605	1200	725	1080	1000	800	wind ⟵
525	1280	405	1400	125	1680	wind ⟶
						boom retracted
1330	470	1330	470	1330	470	no wind
1370	430	1490	310	1770	30	wind ⟵
1290	510	1170	630	890	910	wind ⟶
						boom extended working
555	1300					no wind
595	1260					wind ⟵
515	1340					wind ⟶

Wind blowing along longitudinal axis of wharf

Loads given are the increase in loading per bogie due to the wind loading

Bogie load increase (kn)		Loading condition
upwind bogie	downwind bogie	
−98	98	45 m.p.h. wind
−394	394	90 m.p.h. wind
−1090	1090	150 m.p.h. wind

Worst loading = 1090 + 1330 = 2420 kn
with boom retracted and wind along longitudinal axis.

Horizontal loading

Ship impact:

Assumed to be 2% of the ship's weight i.e. 200 tons = 2000 kn.

Wind loading
Wind blowing perpendicular to longitudinal axis of wharf

Height of top of wharf + 2.5 m of stored material = 7.5 m.
@ 45 m.p.h. p = .238 kn/m² = 5 lb/ft²
@ 90 ,, p = .952 kn/m² = 20 lb/ft²
@ 150 ,, p = 2.63 kn/m² = 55 lb/ft²

Height of ship = 13 m
@ 45 m.p.h. p = .238 kn/m² = 5 lb/ft²
Light draught wind area ≏ 1000 ft²/1000 tons
∴ area = 930 m²

Horizontal forces	@ 45 m.p.h.	@ 90 m.p.h.	@ 150 m.p.h.
on crane	164 kn	656 kn	1800 kn
on wharf	155 kn	620 kn	1700 kn
on material on wharf	78 kn	312 kn	860 kn
on ship	220 kn	880 kn	—

Wind blowing along longitudinal axis of wharf

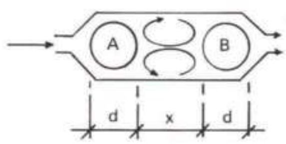
Light draught wind area = 220 m²

Forces	@ 45 m.p.h.	@ 90 m.p.h.	@ 150 m.p.h.
on crane	230 kn	920 kn	2530 kn
on wharf	32 kn	128 kn	352 kn
on material on wharf	16 kn	64 kn	176 kn
on ship	53 kn	212 kn	—

Current loading

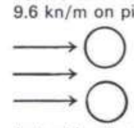
Max current according to Admiralty Bay of Bengal Pilot 10 knots = 5.18 m/sec
Chittagong Port Authority Booklet 8 knots = 4.10m/sec.
Forces on piles are derived from the equation
$F = C_e A \frac{V^2}{2g}$ where C is coefficient which varies with the pile layout.

Longitudinal loading



Cd for pile A virtually constant @ 1.17 for x/d > 3 (for x/d < Cd is less.)
Cd for pile B virtually constant @ .3.

For 10 knot current on 24" pile vertical loading F = 9.6 kn/m on pile A.



Cd virtually constant @ 1.18.

Lateral loading

Cd = 0.13 Ref Apelt & Isaacs Journal A.S.C.E. Hydraulics Division. Jan. '68.
∴ lateral force = 1.75 kn/m depth/m.

Wave forces

Assuming that the wind speed will not be consistently greater than 80 m.p.h. for one hour then these forces are not significant. The most conservative estimate shows these forces to be considerably less than 120 tons with the ship tied up.

Fig. 2
Loading conditions

The sheet piling was to be Larssen No. 5, provided by the client on a free issue basis, with the contractor being responsible for pile driving and protection against corrosion. The wharf site is fortunately protected from the direct effect of tidal waves which have caused devastation in the area on many occasions, but cyclones had to be taken into account as well as tidal and seasonal variations in river level. As a general rule shipping heads for the open sea when a cyclone is expected but it was decided that the clinker wharf should be designed to resist a peak wind force of 150 mph acting on a berthed vessel without the assistance of the storm bollards which were to be provided as separate individual restraints beyond each end of the wharf structure.

The normal berthing procedure is for a vessel to travel upstream on a high tide past the berthing point and then to turn around in mid-river before travelling slowly downstream to berth, the manoeuvre being normally achieved with the assistance of tugs.

River and tidal flows can be quite fierce and a 10 knot current was deemed for design purposes to be the maximum flow near the bank where the wharf is sited on the outside of a large bend in the river.

A summary of the main design considerations is given in Fig. 2.

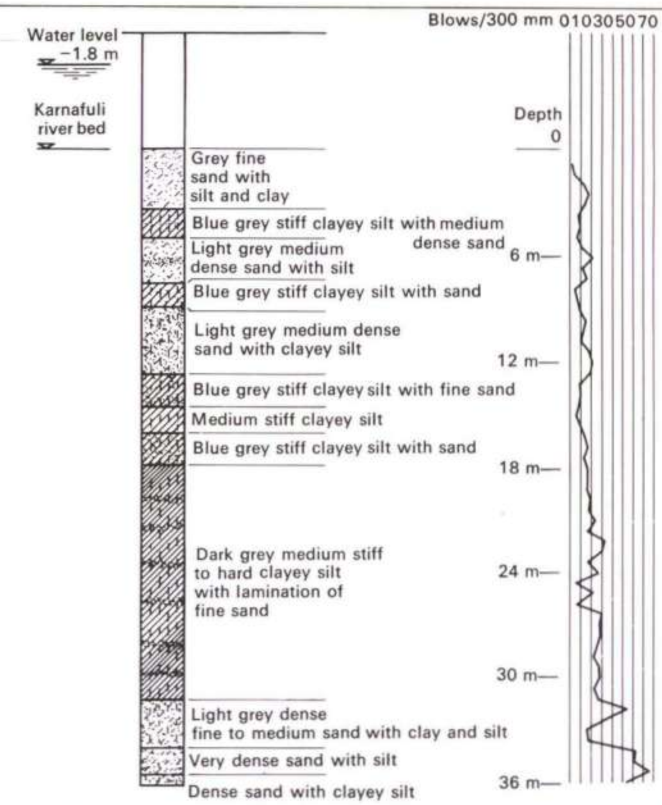
Other recently constructed wharves in the vicinity of the site suggested that a reasonably conventional form of construction using steel or precast concrete piles, approximately 40m long, might present the best answer to the various requirements, but it was decided that we should await the results of the site investigation before being influenced in any way despite the considerable pressure exerted by the very tight programme to get some work under way immediately.

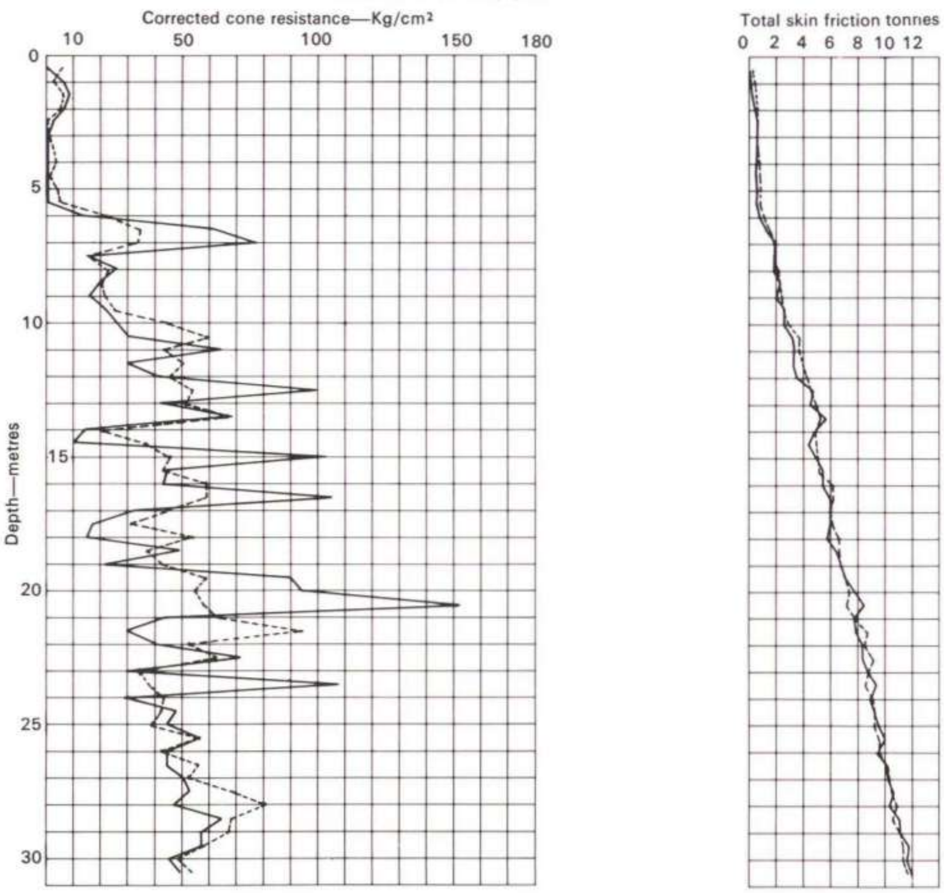
A refreshingly open-minded approach on the part of the consortium concerning the type of substructure that could be used led to particular interest in caisson construction which had been successfully employed up country by The Engineers Ltd. for bridge piers, the material used being either brickwork, of which there is a plentiful supply, or shingle concrete which involves the transportation of large river pebbles from Sylhet some 200 miles away to the north east. (Sylhet is also renowned for its export of culinary skill and it is said that over half of the Indian restaurants in the United Kingdom are run by people from the town or its environs).

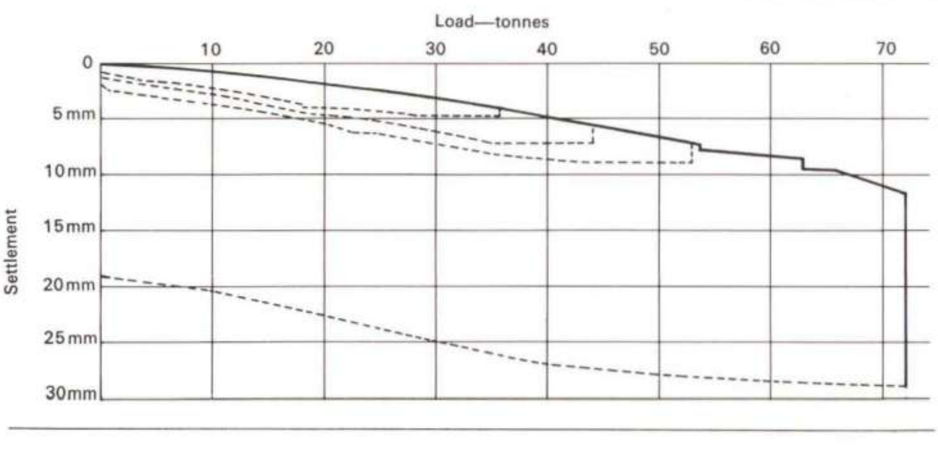
Tests on site

The site investigation work consisted of boreholes, Dutch cone penetrometer tests and the test loading of a 215mm external diameter tubular steel pile with a conical head of the same diameter.

All of this work was carried out on the river bank in mid-1969 and, from the results which are summarized in Fig. 3 developed the substructure scheme utilizing 610mm diameter octagonal precast concrete piles 22m long under the river edge of the wharf and well piles of 3m overall diameter under the bank edge. To resist the outward lateral loading, applied by the bank on the steel sheet pile wall, and the forces produced by berthing and wind, nine anchor wells were introduced within the cement factory site with tie forces carried by







steel cables running under the road and railway to link the anchor wells to the substantial beam at the rear of the wharf structure.

Whereas local experience with well piles had proved their vertical load-carrying capacity there was no reliable information on their lateral anchor strength and we decided to establish this by a full-scale test using two working anchor wells which would be thrust apart by jacks in a direction parallel to the wharf and hence at right angles to the direction in which they would have to resist a thrust when finally incorporated into the wharf construction.

The method of testing is shown in Fig. 5 and the results obtained healthily confirmed the design assumptions.

It was hoped to achieve a maximum horizontal load of 400 tonnes during the test but difficulties with the hydraulic jacks resulted in a maximum applied load of 350 tonnes causing a horizontal movement of 100mm at the top of the loaded anchor wells. Under normal working conditions the applied horizontal thrusts would be very much under 100 tonnes for which the load test gave a top horizontal movement of 5mm which would not be achieved in practice as the wing walls provided at the well tops to develop additional passive pressure were facing in the wrong direction to make a significant contribution during the anchor well test. Under cyclone loading the force would be just over 150 tonnes but this would not be sustained for a long period.

The safe, sustained, vertical load-carrying capacity of each well pile was calculated to be 520 tonnes and the maximum applied loading including wind, 550 tonnes. The 550 tonnes consisted of 258 tonnes dead load, 182 tonnes live load and 110 tonnes from wind, most of the live load and wind coming from the moveable unloader. Fig. 4 shows the site after sinking of the caissons but before pile driving had started.

In view of the fact that the length of the precast concrete piles was much shorter than piles previously provided in the vicinity on similar projects, we decided to test load one of the working piles supporting the outer edge of the wharf by using steel kentledge supplemented by a limited amount of tension (15 tonnes) provided by the four adjacent working piles.

The decision concerning the length of pile relied on the layer of sand at 20m depth and the fact that going to twice that depth did not lead to any worthwhile material by comparison. The chosen pile arrangement of three piles every 4m gives a maximum load per pile of 48 tonnes of which 23 tonnes is dead load.

A 60 tonne test load on the 610mm octagonal working pile gave 4mm settlement with a residual settlement of 1.5mm and 100 tonnes

Fig. 3a top left
Typical borehole log and standard penetrometer test results

Fig. 3b centre left
Typical and average results from Dutch cone penetrometer soundings (Full line indicates typical result and dotted line indicates average result)

Fig. 3c bottom left
215 mm diameter test pile – load/settlement graph

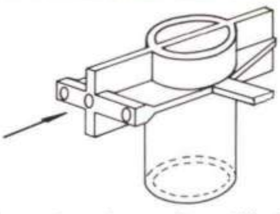
gave 7mm and 2mm respectively so again we felt reasonably happy about our selection of a suitable pile.

To withstand impact from large tree trunks or other objects swept down the river when in flood, a cluster of three precast piles within a large concrete tube was arranged at the upstream end of the front row of outer piles. The cavity between the three piles and the outer concrete tube (formed from precast ring units) was then filled with concrete to form a robust composite member.

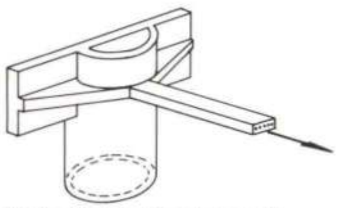
At the mouth of a tidal river what goes down can also come up, so an identical three pile cluster member was also provided at the downstream end of the outer row of piles.

Particular consideration was given to the stresses that could be induced in the piles after driving, when they would be acting as unrestrained cantilevers prone to oscillations caused by and in the direction of the river flow. At the time detailed research was being carried out by CIRIA following surprise damage to circular steel piles during construction of a major terminal at Immingham, but information currently available at the time indicated that this mode of action would be within acceptable limits if tops of piles were braced together temporarily whilst in the cantilever condition.

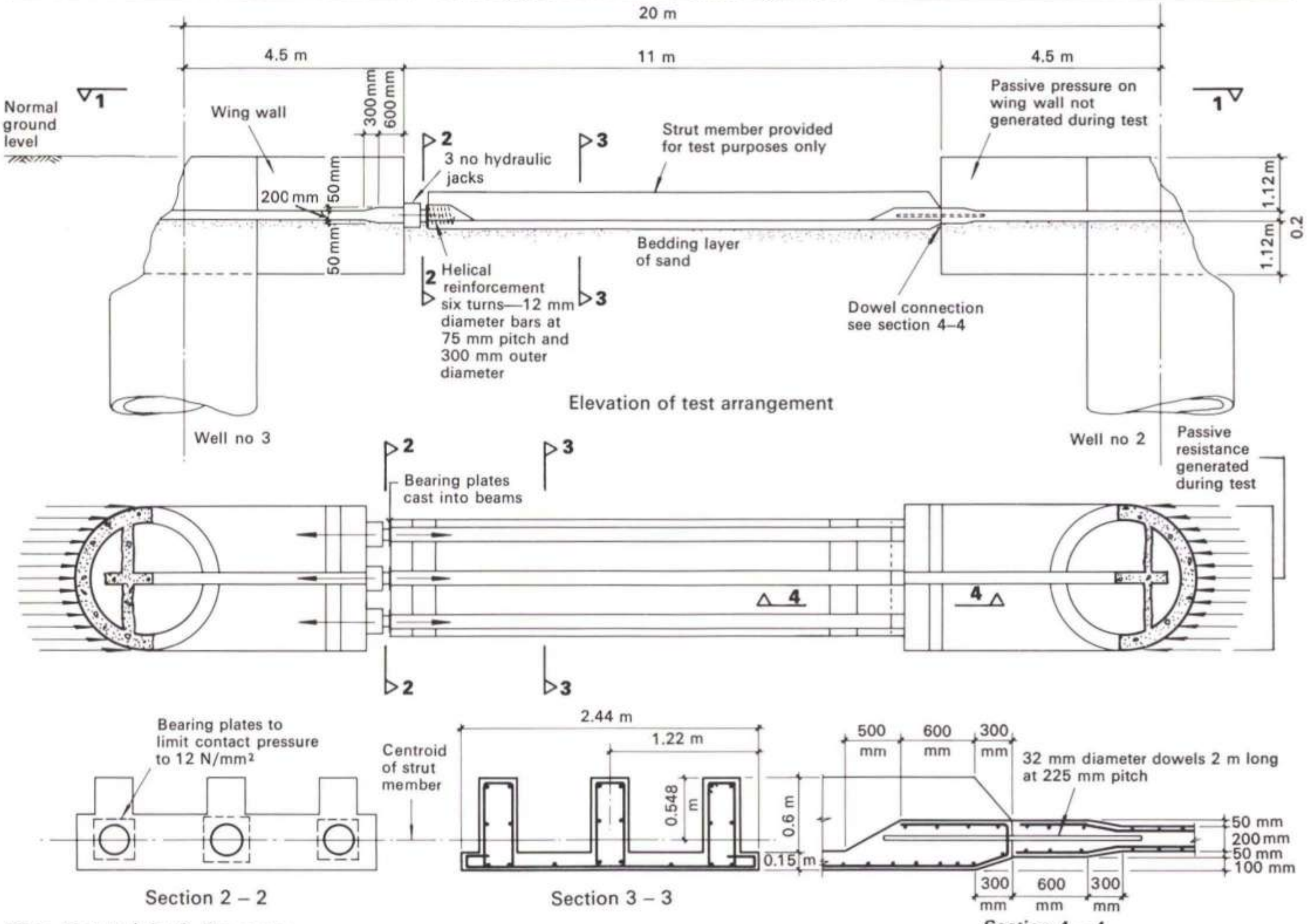
Fig. 4 right
The site after sinking of well piles
(*Photo:* Ove Arup & Partners)



Head of anchor well modified for lateral load test



Typical head of anchor well



Figs. 5a top right **& 5b** centre
Lateral load test on anchor wells

Steel sheet piling

The steel sheet piling developed into a more complicated issue than was anticipated at the outset, as the client decided in the interests of keeping imported steel to a minimum, to make do with what was already available within the eastern and western wings of the country.

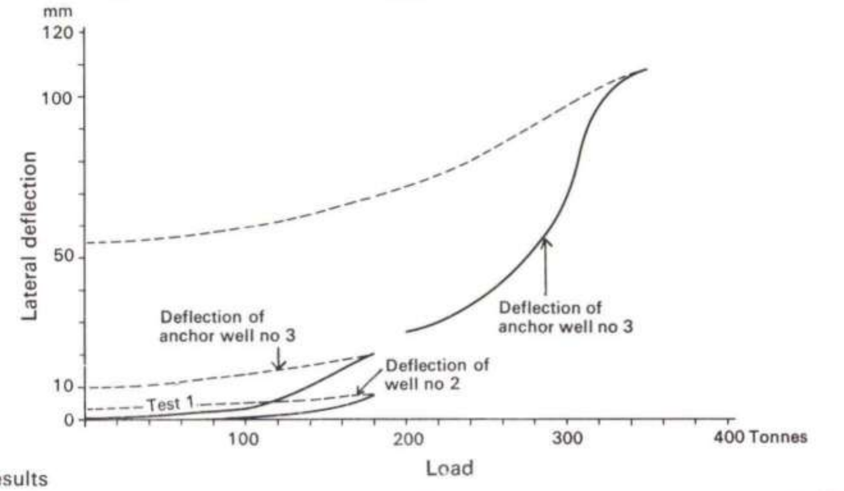


Fig. 5c
Load test results

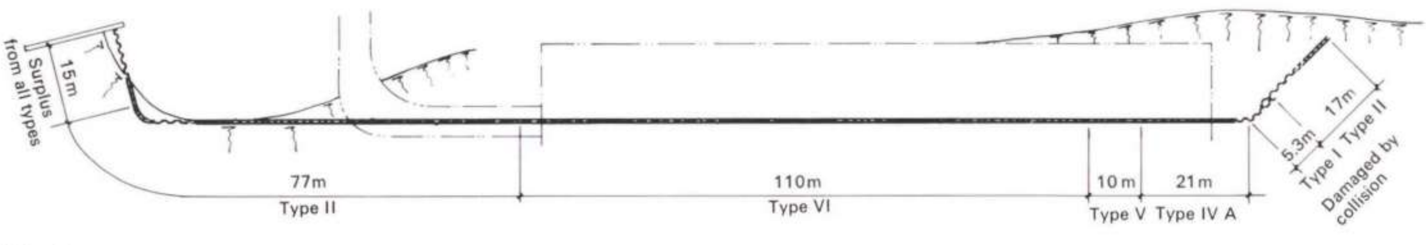


Fig. 6a
Arrangement for steel sheet piling indicating use of different profiles

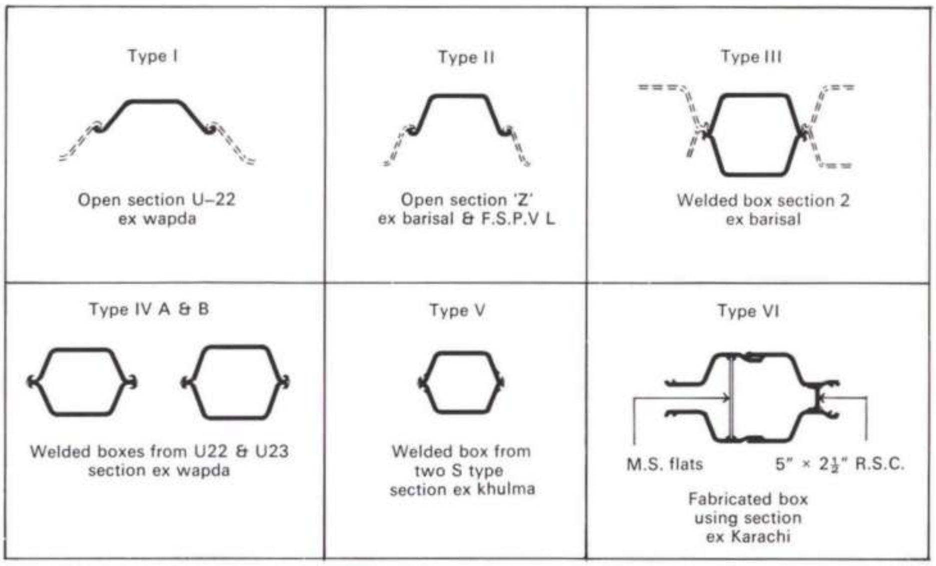


Fig. 6b
Steel pile profiles

This led to a nationwide search by the Chief Engineer of EPIDC and our joint visit to Karachi in December 1969 to seek out suitable supplies. The official sources who were approached were unable to provide any sizeable tonnage to match the required quantity of almost 1000 tonnes but several hundred tonnes of a light section were offered and accepted as it was clear that a fabricated box section could be made up on site to give the strength of walling required.

A further search in the metal merchants quarter of Karachi, where scooter rickshaws and taxis dart in between the camel-drawn carts, was less successful but later on small quantities were located in the eastern wing and Fig. 6 illustrates how eventually a variety of different sections was put together to form a total wall length of over 250m.

The design of the wharf deck was influenced by the fact that normal formwork could be used on the bank side of the steel sheet pile wall where land access was relatively straightforward, whereas on the other side of the sheet pile wall one would be working over swift flowing water of several metres depth. This led to the use of precast prestressed concrete beams supporting precast concrete permanent formwork panels for the outer half of the deck, the topping being in situ reinforced concrete.

Consideration was given early on in the design stage to the choice of including or excluding the steel sheet pile wall from carrying vertical loads from the deck and, as the latter would have involved the complexity of a top connection detail capable of transmitting horizontal forces but not vertical ones, we decided to adopt a fixed connection and include the wall as a significant part of the vertical load-carrying structure. Predicted settlements could not be relied on to indicate what would happen under the highly variable live load conditions applied to three differing types of piling, (610mm octagonal precast, steel sheet and wells) each having a different length of embedment in a different part of the cross-section of the river bank, so the in situ beams on the bank side of the sheet wall and the precast beams on the other side were both designed as simply supported with additional steel reinforcement placed in the tops of the beams over the central line of support formed by the sheet pile wall.

This deck design also suited the construction method of completing that half of the deck nearest the river bank first so that it could support the mobile crane to be used in placing the precast units forming the first stage structural elements on the other half.

A single expansion joint was provided in the middle of the deck structure.

Fendering

The design of the fendering system was also primarily influenced by the clear-cut need to utilize what rolled steel sections were available in the east wing of the country at the time and a 460mm deep beam section set the upper limit on bending strength.

Some time after design work was under way the Chittagong Port Trust laid down the requirement that the fendering system should prevent small vessels 6.1m long from passing at all states of river level.

These two factors led to a comparatively complicated system incorporating Goodyear cylindrical rubber fender units at both deck level and some 4.5m lower down so that the main slightly raked fender piles of twin 300mm Universal Column Sections would not be stressed to a permanent set condition under vessel berthing impact applied when the river was at its lowest level.

Having then arrived at a system which to all obvious appearances seemed to provide a soft berthing, we became mindful of the general rule that the care exercised when berthing a vessel against a wharf tends to be inversely related to the resilience of the latter because of the likelihood of damaging the vessel. We therefore decided that it would be wise to make some allowance for an occasional higher approach speed.

We then reconsidered the permanent set condition of a raked fender pile and decided that it could still partly fulfil its purpose provided its deformed state did not result in a panel for the fendering projecting out in front of the fendering line, where it would be particularly prone to berthing forces parallel to the river. Restraint against this possible movement was easily arranged by rebound chains connecting the top reinforced concrete fender beam to the deck itself and furthermore by connecting the ends of each individual panel of fendering to its adjacent panels.

The final arrangement of the fendering and typical details are given in Fig. 9.

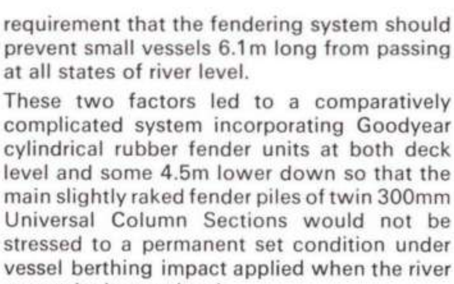


Fig. 7
Pile casting yard adjacent to Khal 7
(*Photo:* Ove Arup & Partners)

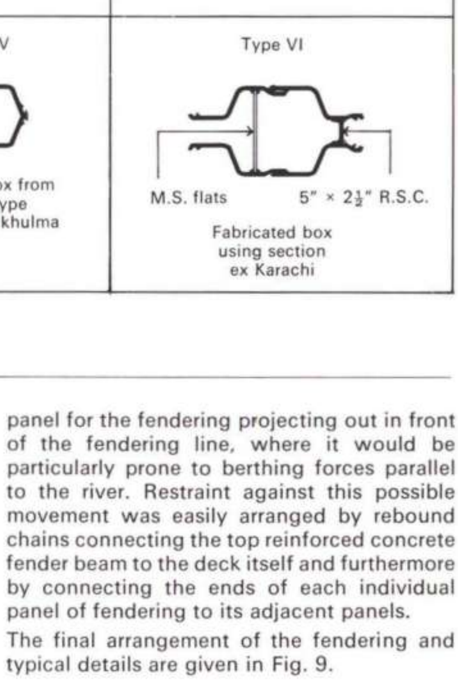


Fig. 8
Pile driving with pile-carrying pontoon
in foreground (*Photo:* Dick Wright)

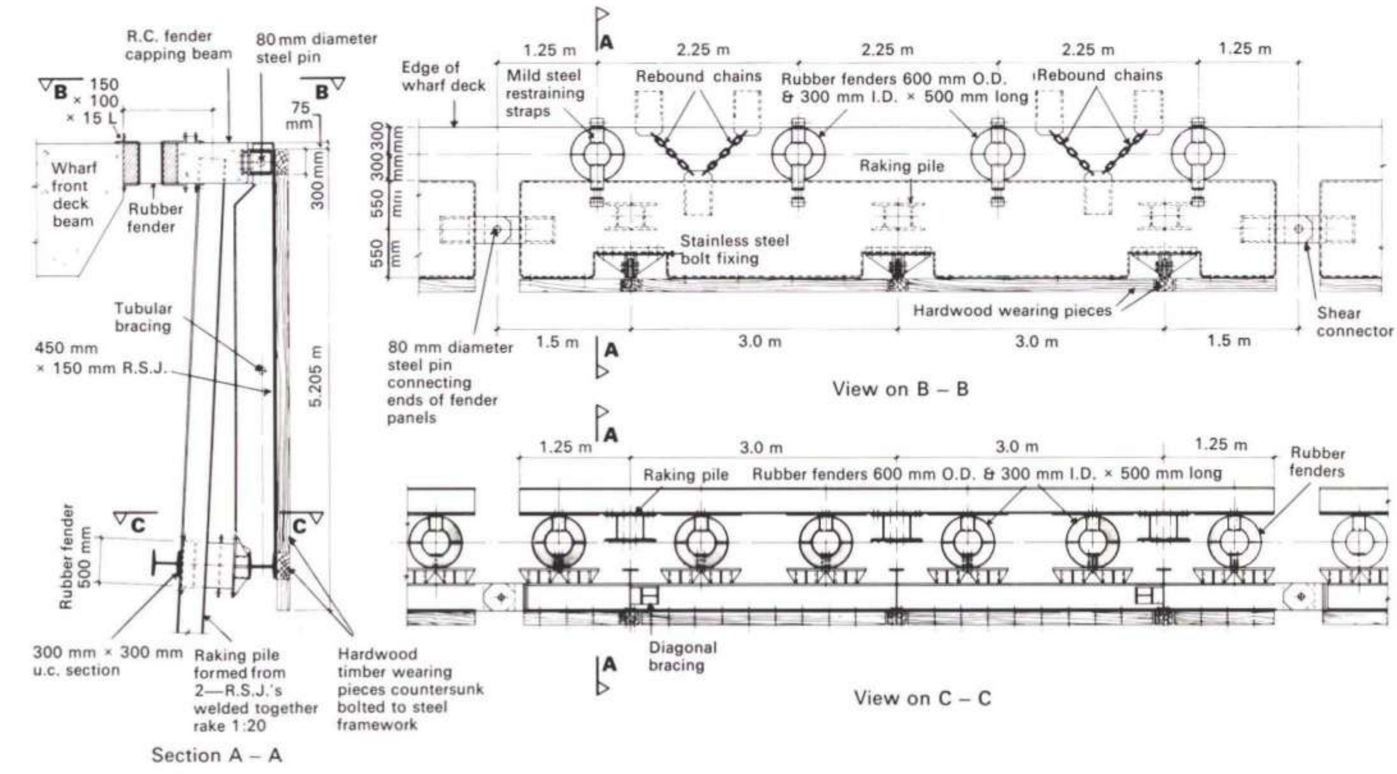


Fig. 9
Detail of typical fendering panel

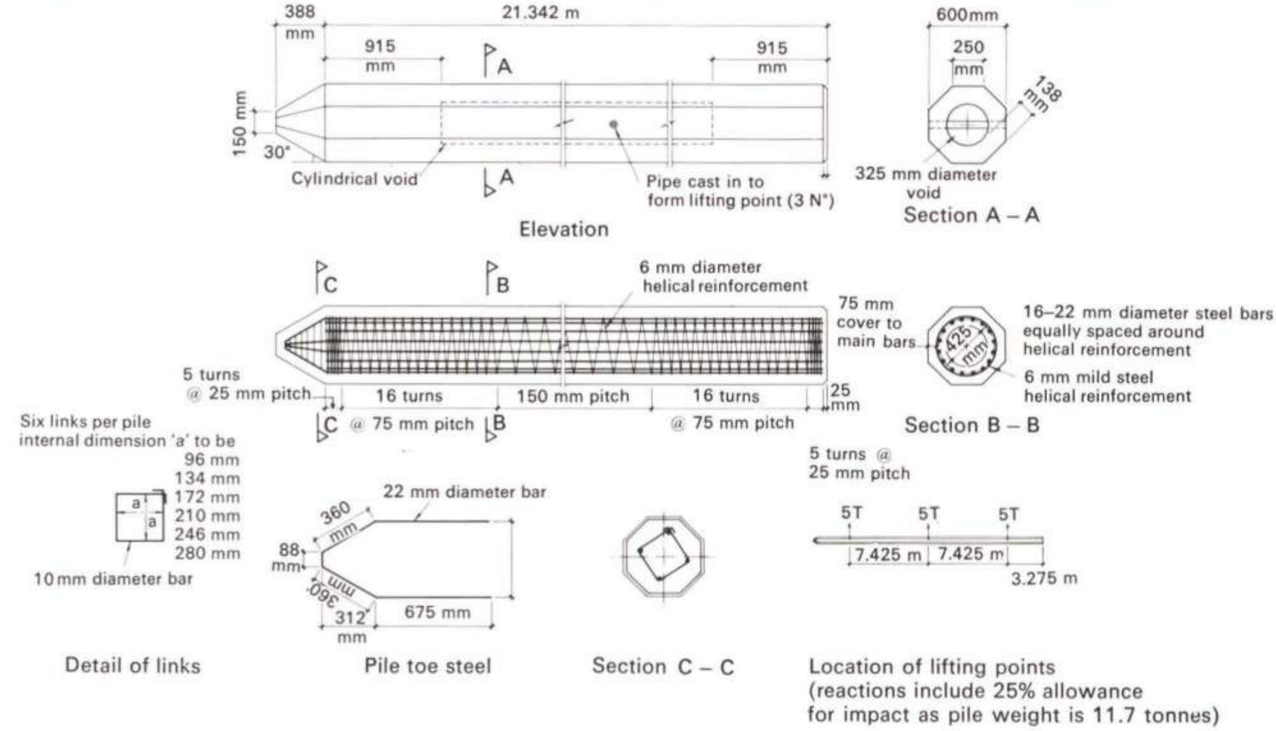


Fig. 10
Detail of 600mm octagonal reinforced concrete pile. Minimum crushing strength of concrete 35 N/mm² at 28 days

The design of the fendering assumed a maximum approach speed of 240mm per second, and an impact length of 22m of fender which results in energy absorption of 16 tonnes m. for a deflection of 230mm excluding the relatively insignificant effect of hydrodynamic mass. The fender reaction was calculated to be 240 tonnes.

Construction stage

Once the decision on the type of precast concrete pile had been taken, preparation work commenced on a pile casting yard adjacent to a stream known as Khal 7 feeding into the Karnafuli river just below the wharf site.

This preparatory work included driving timber piles into the stream bank to help form a level casting yard and to increase its plan area. Brick soiling was then laid on the levelled ground to form a pavement and pile casting commenced utilizing the purpose-made steel moulds.

Raymond International were awarded the sub-contract for driving the steel sheet piling and the 610mm octagonal precast piles, and they mounted their rig on a barge having a 1.2m draught, the hammer energy being 2,800 kgm. Once the piling rig was ready to start driving, the first few piles were moved transversely one by one across to Khal 7 where they were placed during high tide on a buoyancy frame made up from empty oil drums. See foreground of Fig. 8.

Fig. 11
Barge mounted piling rig
(*Photo:* Dick Wright)

Fig. 12
Assembling the rear beam reinforcement cage (*Photo:* Ove Arup & Partners)

Fig. 13
Upstream pile cluster protection (collision damage to sheet piling on right) (*Photo:* Donald Holliday)

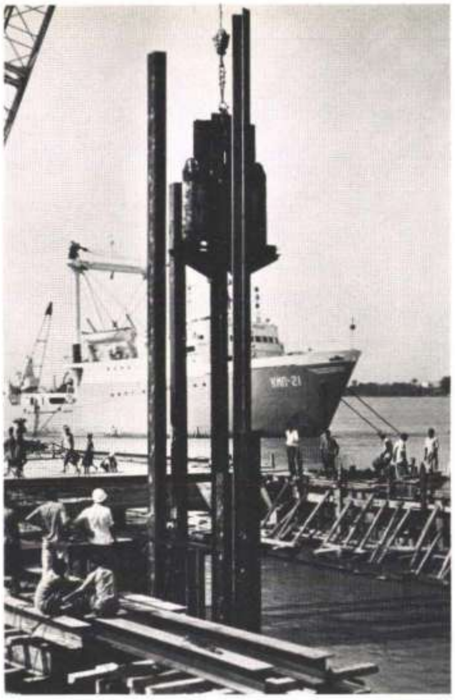


Fig. 15
Driving the steel sheet piling (*Photo:* Ove Arup & Partners)

Fig. 17
The mechanical unloader being assembled (*Photo:* Geoffrey Wood)

Fig. 18
The wharf in use (*Photo:* Trevor Honybun)

Fig. 14 below
Fender panel steelwork (*Photo:* Geoffrey Wood)

Fig. 16 right
Fendering after completing three panels (*Photo:* David Brunt)

This frame was then floated down into the Karnafuli and towed upstream to the piling barge where the pile was hoisted vertically by the rig and positioned for driving.

The first few piles driven during February 1971 were intentionally driven well into the sand layer to test the strata, and from the blow count it was found that 1-1.2m penetration into the sand gave the maximum resistance.

The sinking of the well piles was carried out whilst the precast piles were being cast and before any piles were driven, the caissons had reached formation level and the bottom concrete plugs had been cast in place. The major beam connecting the well piles was then cast on top using ground made up to the correct level as a soffit shutter.

Well piles were also sunk for the storm bollards as by this time the advantages of caisson construction were well proven on the site, the form of construction calling for very little equipment other than a simple trussed timber hoist, pumps and normal concrete mixing plant. All excavation within the wells was carried out by hand and there was no need to resort to jetting for which vertical pipes had been cast into the first few well piles constructed.

At first the well piles tended to tilt slightly when sinking but very quickly this was controlled by excavating internally on the side opposite the direction of tilt and applying correcting forces externally at the top, using cables and either turnbuckles or a Spanish windlass.

Then in November 1970 the country was hit by a cyclone that killed many thousands and left millions homeless. Chittagong was given warning in good time and in any case was not close enough to the eye of the cyclone to sustain severe damage but in April 1971 all work came to a halt as a result of the civil strife that led to the creation of Bangladesh.

In February 1972 the Bangladesh Industrial Development Corporation, the government body that had taken over the client role, requested an early resumption of work because of the vital need for dock facilities to handle the priority material so desperately needed by the new nation.

At the cessation of hostilities Chittagong port contained a number of sunken wrecks but the clinker wharf was lucky to have sustained only slight damage, this being incurred by the upstream return length of the sheet piling which was parted at a point near the pile cluster by the impact from an uncontrolled ship. See right hand side of Fig. 13.

The consortium had lost a lot of vehicles and plant during the conflict and a labour dispute also hampered an immediate resumption of work, but by mid-August 1972 all concrete piles had been driven and, following the subsequent monsoon season, steady progress was maintained until all the work was substantially completed in 1974. Installation of the system for the cathodic protection of the steelwork by the sacrificial anode method has been delayed but should be completed by the time this article appears.

Credits

Client:
Bangladesh Mineral, Oil and Gas Corporation
Main contractor:
Joint Venture – Stoneville Engineers Ltd. and The Engineers Ltd.

Lagoon Barriers at Venice

Alistair Day
Peter Rice

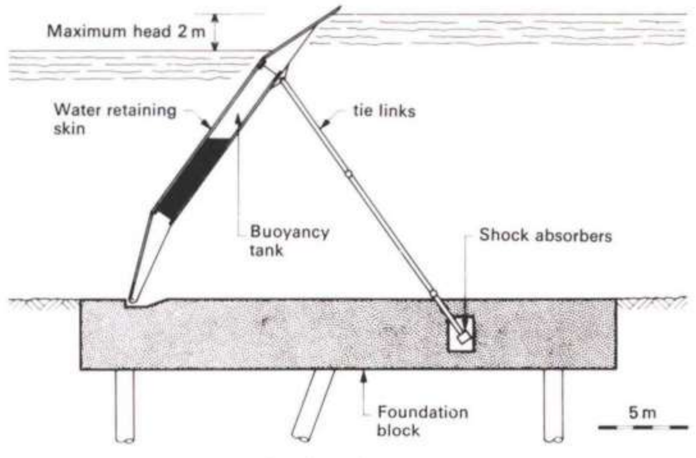
The islands on which Venice was built are very low lying and excessive extraction of water from aquafers below the islands has caused a general lowering of the ground level, to such an extent that there is serious flooding of the city whenever a very high tide occurs. The tides which cause the flooding occur quite frequently and when we heard that a competition had been organized to find a scheme to prevent this flooding, the firm decided to work up an idea which had been started in another context.

Venice is in a lagoon formed by a string of islands separating the lagoon from the Adriatic. There are three passages from it to the sea and one of the objectives of the competition was to find a proposal to control flood waters passing into the lagoon from the sea.

The Venice idea was developed as a scheme for a competition but unfortunately we did not know when we started that the competition was more in the nature of a design and construct tender, not a competition in the accepted sense. This meant that unless the scheme was taken up by an Italian contractor, there would be no chance of it progressing, so that it became necessary to attempt to discuss it with some of them. Renzo Piano of Piano and Rogers very kindly arranged a discussion with a contractor and with Mr. David Trevisani of the firm of Trevisani Piling. We are very grateful to Mr. Trevisani for his efforts on our behalf but as far as we can tell the main reason we did not get anywhere is the fact that the Italian contractors had been working on the scheme for a very long time, even before the competition had been started, and those who were going to enter had already decided on the designs they were going to submit. We were in effect entering the competition far too late.

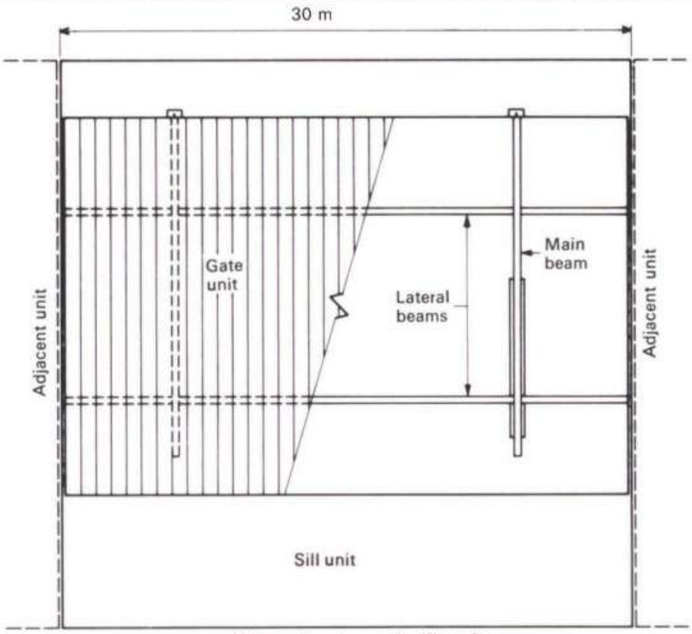
The problem was unusual as the main operational requirement was the necessity to close large gates in a rapidly flowing waterway. Normally large lock gates are positioned in still water, for example in slack water at high tide, or else there are a pair of gates which are opened in turn to allow a ship to pass.

Being an unusual problem, an unconventional solution may have been needed, which was a possible reason for looking for alternative proposals from a competition.



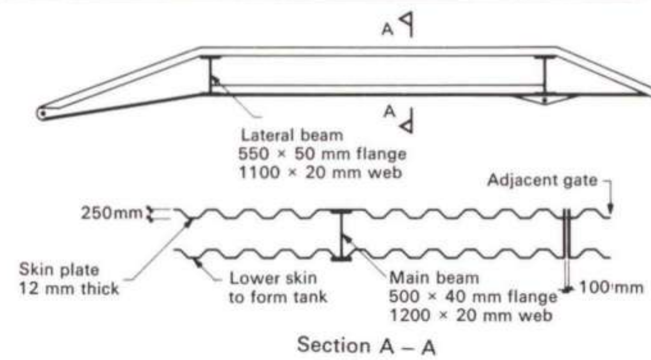
Section of gate

Fig. 1
Principle of the barrier



Plan of gate and sill unit with gate lowered

Fig. 2
Skin supported on two main beams



Section A – A

Fig. 3
Detail showing member sizes required to resist maximum load

Development

The scheme which was developed is shown on Figs. 1 and 2. Pressed steel troughing was to form a skin which would be the water retaining member. The skin would span onto two lateral beams which were to be supported on two main beams as shown on Fig. 2. The details on Fig. 3 show the member sizes which are required to resist the maximum load which can occur. When the gate was closed the horizontal load due to the differential water level would be transferred to the foundation by the main beam acting as a strut held in position by the tie member. The horizontal water force would act at a level above the foundation block so that sufficient weight would have to be provided in the foundation to make the resultant of the horizontal and vertical forces pass through the base of the foundation.

The main factors governing the development of the scheme were the operational requirements and the cost, constructional and maintenance considerations.

First the operational requirements. When not in use the gates would rest on the bed of the channel so that means of raising them to their operational position would have to be provided. Mechanical devices such as hydraulic rams were a possibility but a much more efficient and cheaper method would be to raise the gate by providing buoyancy tanks in it. Water is very heavy and by displacing it large forces can be generated. By providing a second skin below the beams a tank could be formed in the gate and, when emptied, the buoyancy of the tank could easily lift the weight of the gate, together with a thick layer of silt on it. It is highly unlikely that the silt would accumulate, because a structure such as this in the bed of a channel tends to promote scour, not silting. This is because it is smoother than the channel bed so that the water moves faster over it, picking up energy, which then has to be dissipated by scouring the rougher bed.

Compressed air would be used to dewater the tanks because it was found that standard air compressors would provide the amount of air needed to raise the gates in about 10 minutes. This means that no specialized machinery would be required. The compressors could be housed on shore with the air piped to the tanks, so that no piers are required in the waterway.

The sequence of closing the waterway would be as shown on Fig. 4. The buoyancy of the gate would carry it to the surface if it were raised at slack water. However, it was essential that the gate could be closed at any state of the tide, even at half tide, when the current is at its maximum. In this case as soon as the gate is partially raised and begins to affect the current it would be lifted up and rapidly raised to the surface. By the time it reaches the surface it would be moving at practically the speed of the water and would pick up a considerable amount of kinetic energy. In addition when the gate stopped a hydraulic jump would immediately form on both sides of the gate, producing a differential head which could be a significant part of the full design head.

The crux of the design lay in stopping the movement of the gate. If the movement was stopped abruptly when the links of the tie straightened out, there would be a large dynamic load factor producing excessive forces in the structure. It was not possible to make a viable design unless the maximum force in the tie could be reduced to a value comparable with the maximum static force required to resist the differential head. It was the discovery that there were commercially available shock absorbers of just the right capacity to produce a retarding force less than the final load in the tie, which established the feasibility of the proposal.

To lower the gates the pipelines which supply the compressed air to raise them would be opened to the atmosphere. When the water on the seaward side fell below that on the landward side, the gates would begin to sink with the air in the buoyancy tank now venting to the atmosphere. The gates would be heavy so that a pressure would build up in the buoyance tanks to drive the air out. As the gates approached the sill the water trapped between the gate and the sill would be forced out very rapidly causing a turbulent flow of water which would lift any silt lying on the sill and drive it clear. In effect the gate would be self-cleaning when closing. However, to ensure that there would be an adequate outflow of water, supplementary cleaning jets would be incorporated.

Factors in design

One of the attractions of this configuration was the fact that the water load would be carried to the foundations by the most direct path possible. This can be contrasted with a gate between piers, where the force from the water is first carried horizontally to the piers and then down into the ground. For gates of the size necessary to allow ships to pass, the span between the piers would be at least 60–70 m, and the bending moments set up in the gate over this span, by the very large forces, are one of the main factors in the design. In the alternative proposal, the spans where forces are carried in bending of members are kept to an absolute minimum and this has a significant effect on the amount of material required in the gate. In addition, the absence of piers appears to be a very attractive feature. With a navigation channel of 200 m and a total opening of about 1 km in the largest of the entrances to the lagoon, there would be about 12 piers which would be a considerable hazard to shipping.

An important factor in the design of this scheme was the fact that a barrier and not a dam is required. The gates would only be raised for half a tide at most, and if during this period a certain amount of water leaked through the gates, it would have an insignificant effect on the level within the lagoon. This meant that the gates could be considered as independent units and a generous gap allowed between them, which would have an important effect on the cost of fabrication as the tolerances would not be stringent. In addition, the alignment of the sill and gate units would not be critical because when the gates were raised, if they were not exactly in line, the space allowed between the gates would prevent them clashing. The considerable latitude available on the non-alignment would be very valuable in this type of marine construction.

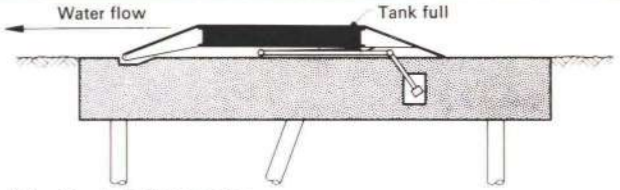


Fig. 4a (a) Start of dewatering

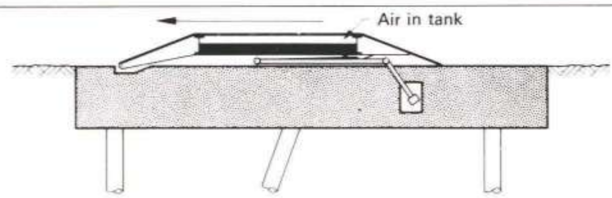


Fig. 4b (b) Tank half empty, gate just buoyant

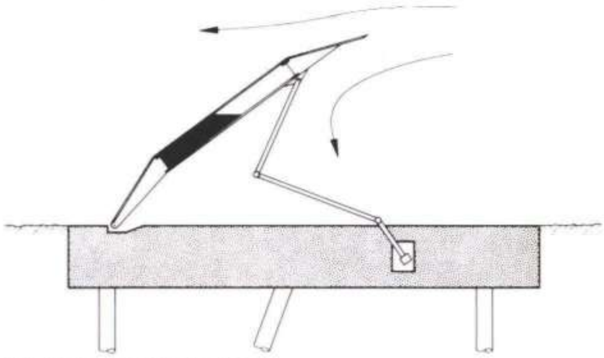


Fig. 4c (c) Gate starting to rise

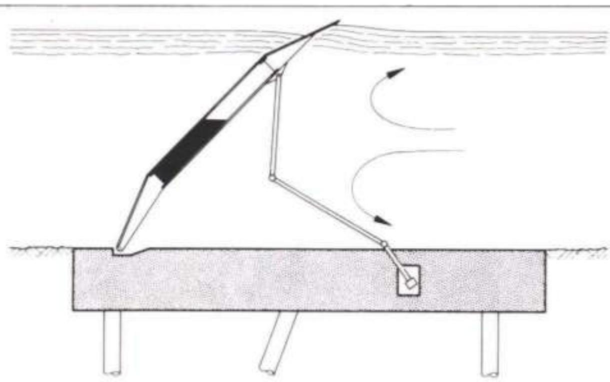


Fig. 4d (d) Gate at surface

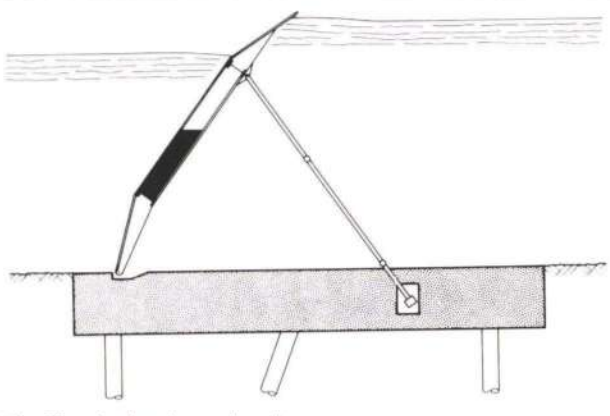


Fig. 4e (e) Shock absorbers closed

Figs. 4a to 4e
The sequence of closing the waterway

Making the gates as a series of independent units, would allow for very easy maintenance. The gate units would be attached to the sill by only four pins and these could be easily demounted and a whole gate removed for maintenance. With the number of gates which would be required it would be a practical proposition to have a spare gate which would replace the one which had just been removed. This could then be maintained without any urgency to replace it in case of another high tide occurring.

When the tenders are finally awarded, it will be very interesting to see which of the many contending schemes will be chosen, and whether any of the features in this scheme appear in it.

Credit Lyonnais: Queen Victoria Street Development

Michael Courtney

Introduction

By 1972 the London headquarters of the French Government-controlled bank, Credit Lyonnais, had been in Lombard Street for a long time, but due to prospective redevelopment they had to move by June 1976. It seems incredible now, but at the time there was an intense shortage of good office accommodation in London and rents were consequently high. Credit Lyonnais decided that it would be advantageous to them to join the property development boom and construct their own building, provided they could secure the right site in the City of London. This they did by obtaining a lease from Wates Property Company on part of a triangular island site in the City by Mansion House station between St. Paul's and the Bank of England.

The consultants

It was at this stage, at the end of January 1973, that the architects, Whinney Son and Austen Hall, were appointed by Credit Lyonnais followed by the appointment of the professional advisers, Tilney, Simmons & Partners, service engineers; Wicksteed, Son & Few, quantity surveyors, and Ove Arup & Partners as structural engineers.

The site

Part of the site included that of the Wren Church, St. Mildred's; destroyed by bombs during the last war, and this section had been cleared ready for development.

Another part of the site was occupied by a fire station which was scheduled to move to occupy premises at Mondial House, the new London Telecommunications Centre. The timing of this move was dependent on the completion of Mondial House and the date for this uncertain.

The third part of the site was owner/occupied and Credit Lyonnais were trying to buy the freehold, but were finding it difficult to reach a reasonable price.

The site had considerable environmental and communication advantages and a building on it would, if strongly modelled, be remarkable. However, the largest possible development on the site, in terms of plot ratio and the existing office development permit, would be much too large for Credit Lyonnais alone. Banco di Roma and Commerz Bank, Stuttgart were therefore invited and agreed to share the space by renting part of the finished building.

Geology

Geological records indicated that the site had some 4 m of uncertain fill overlying approximately 4 m of sand and gravel above a very thick layer of London clay, with a perched water table in the gravel just above the clay. A later site investigation confirmed this hypothesis.

Constraints on design

Due to the proximity of St. Paul's there were maximum height limitations on the building; not just one but a series of levels arranged on a square grid, which was not parallel to any side of the site; and the general level of the ground water was not allowed to be altered, even temporarily, because of the influence on the cathedral's foundations.

London Transport also imposed constraints. The Metropolitan and District underground lines run in cut and cover construction just the other side of Queen Victoria Street. British Rail's Waterloo and City tube line runs beneath the centre of Queen Victoria Street.

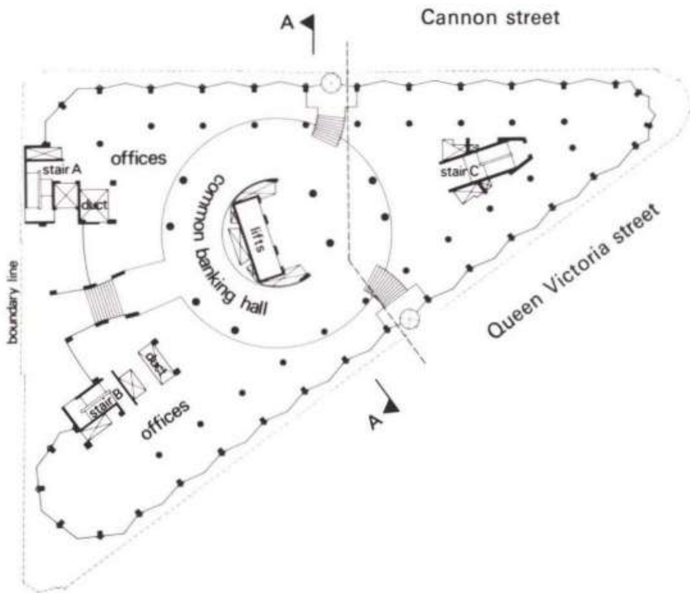


Fig. 1
Ground floor plan

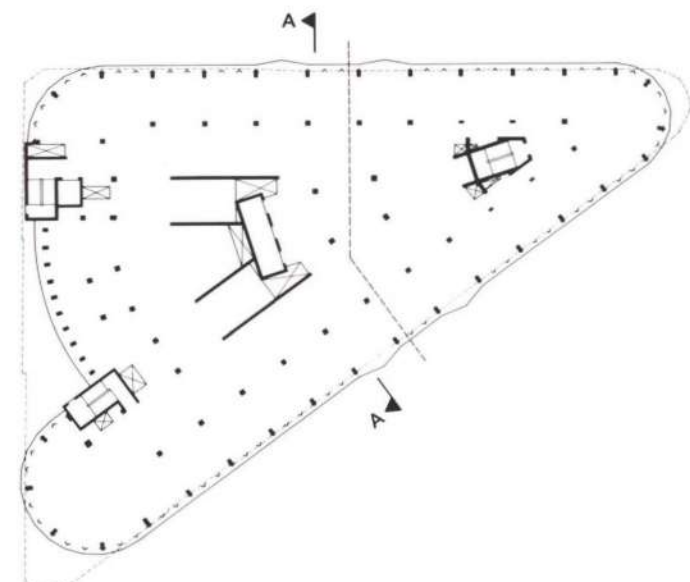


Fig. 2
Typical office floor plan

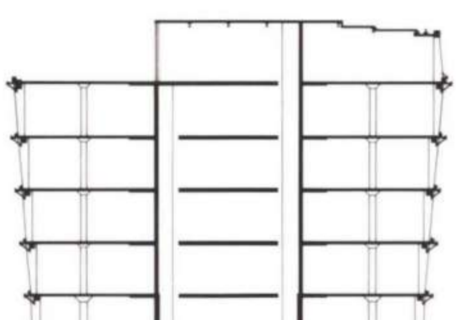


Fig. 3
Section A-A

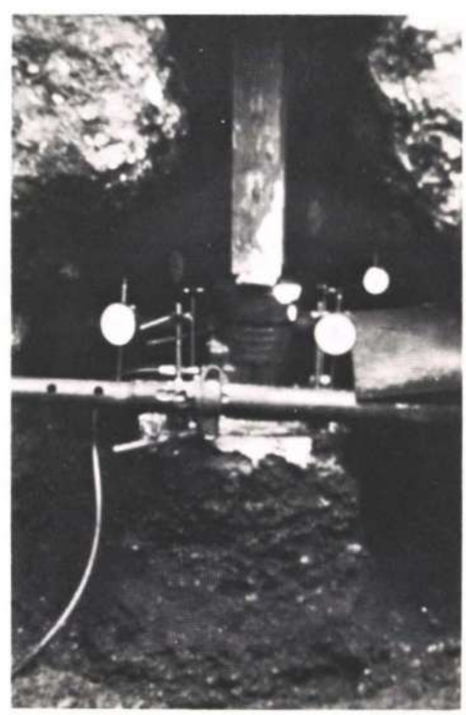


Fig. 4 right
Plate loading test on unconfined cube of gravel cut from chemical grout treated zone (*Photo:* Michael Courtney)

Fig. 5
Phase 2 site after demolition of fire station building in March 1976 (*Photo:* Michael Courtney)

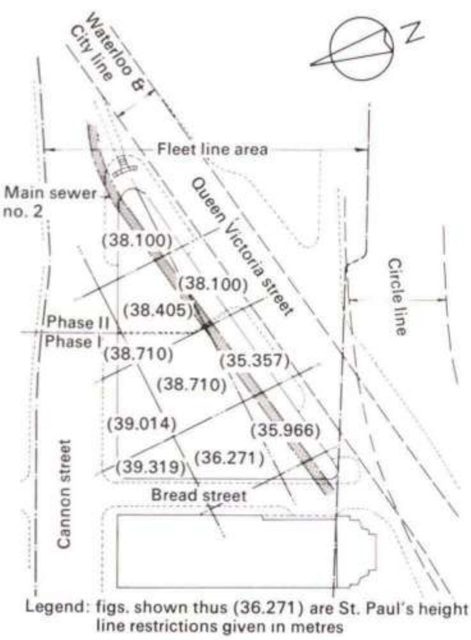


Fig. 6
Plan of site with constraints

Fig. 7
Existing fire station building showing steel tie back restraint at left corner (*Photo:* Michael Courtney)

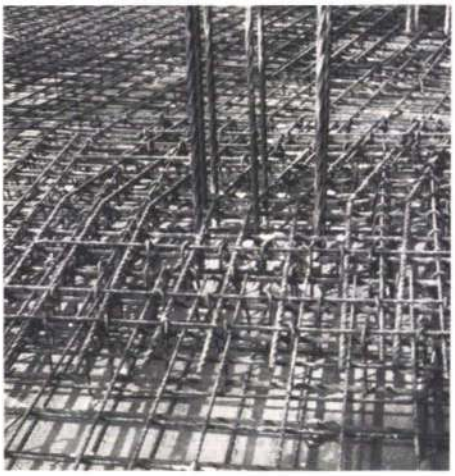


Fig. 8
Shear link reinforcement round column heads (*Photo:* Michael Courtney)

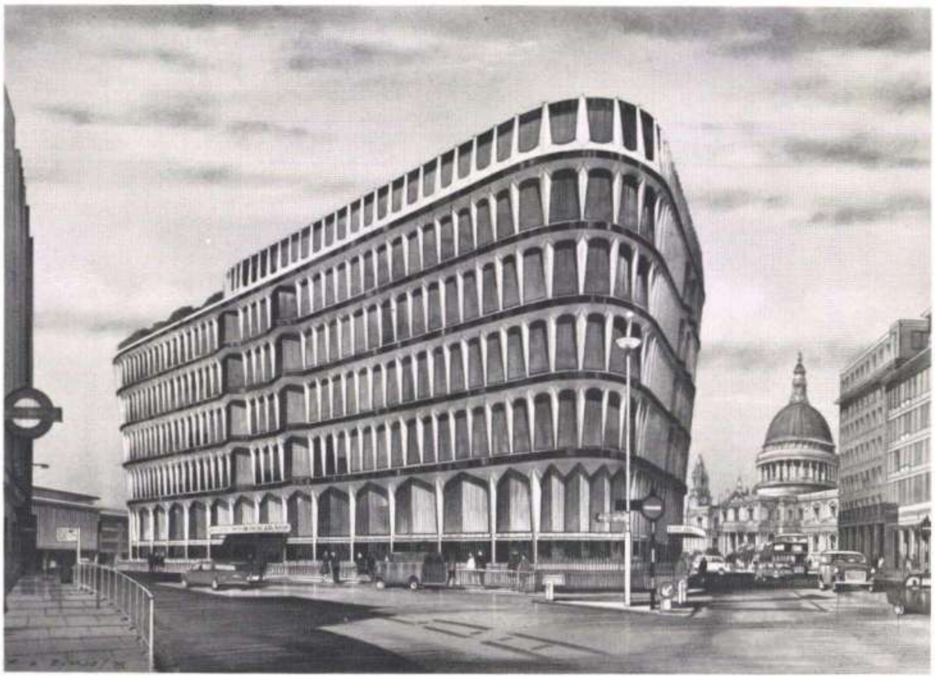


Fig. 9
Credit Lyonnais finished building from east (Artist's impression 1975)

The deviation limits, within which no obstruction may be placed to the future passage of the Fleet line, neatly encompass the site, bringing problems not only of foundation but probable future settlement.

There is also one of the three main London outfall sewers about 15 m directly below the site. These were designed in Victorian times to run at 30% capacity. Now they always run at 50% capacity and sometimes they run full and under pressure.

A further complication was the necessity to provide lateral support at all times to the flank wall of the fire station building. Like so many crosswalls in London's old buildings, it had never been secured to the internal floors.

There were, of course, various other service subways, ducts and passages in the pavement and the roads.

The design

Wates Property Company had already, as prime lessees of the site from the freeholders, the GLC, obtained an office development permit on the basis of a speculative design produced by their architects. The agreement Credit Lyonnais made with Wates Property Company placed restrictions on the development. Wates Construction Company had to construct the building, all details of the building had to be approved by Wates Property Company, and the planning of the building had to provide the same net lettable area as the original speculative design.

The first scheme

The stipulation of minimum lettable area was unfortunate as the building for Credit Lyonnais was intended to be more highly serviced and with a greater headroom between finished floor and ceiling than the speculative design. This meant that the storey height had to be increased from 3.2 m to 3.5 m to provide a clear headroom of 2.8 m with the consequent loss of a whole floor due to the St. Paul's height limitations. The shortfall in lettable floor area was made up by allowing the upper floors to project over the pavement, the projection increasing with each floor until the uppermost floor projected almost 4 m directly over the kerbline.

The problems of the sewers, the tube lines and the water table restrictions were overcome by designing the building on a raft foundation, allowing only a very small increase in the effective pressure on the ground. The excavation was to be carried out within a bentonite-formed, diaphragm cut-off wall so as not to affect the ground water. To achieve speed of construction it was hoped to use ground anchors to stabilize the cut-off wall, but this proved impossible due to congestion of services.

The superstructure of the scheme envisaged large prestressed precast beams on a 1.5 m office module spanning from the central cores to, and over, a perimeter edge beam. The central cores would be linked by two large beams parallel to Queen Victoria Street and Cannon Street to form a continuous internal support, and the oversailing of the pavement was to be accomplished by the continuous precast beams. The hogging moments produced by the oversailing areas were to be equalized by vertically linking the beams together within the stepped façade, thus reducing the largest moment to be carried.

The services would be zoned within the 1.5 m module formed by the precast beams, with the flow and return pipes running between them and passing through preformed holes in the internal supporting beam. The central area contained by the two cores and the two large internal beams was beam-free to provide the longitudinal service distribution area. The air-conditioning chosen by the services engineer was a variable volume, dual duct system, although the possibility of using a fan coil system was discussed.

At the end of February 1973 application for planning approval was made for this scheme, discussion with the planning officers having indicated that the oversailing solution was acceptable, provided the whole site was a comprehensive development.

This scheme was developed during 1973 with increasing difficulties in incorporating the air-conditioning ducts and increasing need for plant room space which meant that the basement became deeper and deeper. By October 1973, when the building was costed, the spacing of the precast beams had been increased to 2.1 m centres and there was effectively a three-storey height basement which was giving some concern due to the proximity of the main sewer. The large projections over the pavement had also been lost as Credit Lyonnais refused to pay the asking price for the corner area, but a sloping façade had been retained at the request of the planning officers.

The second scheme

Late in November 1973 a consultants' meeting was called in the architect's office following his return after reporting to Credit Lyonnais in Paris. The client had objected to the 2.1 m module and the cost of the building; it had to be revised on a 1.5 m module to a budget cost reduced by £1m.

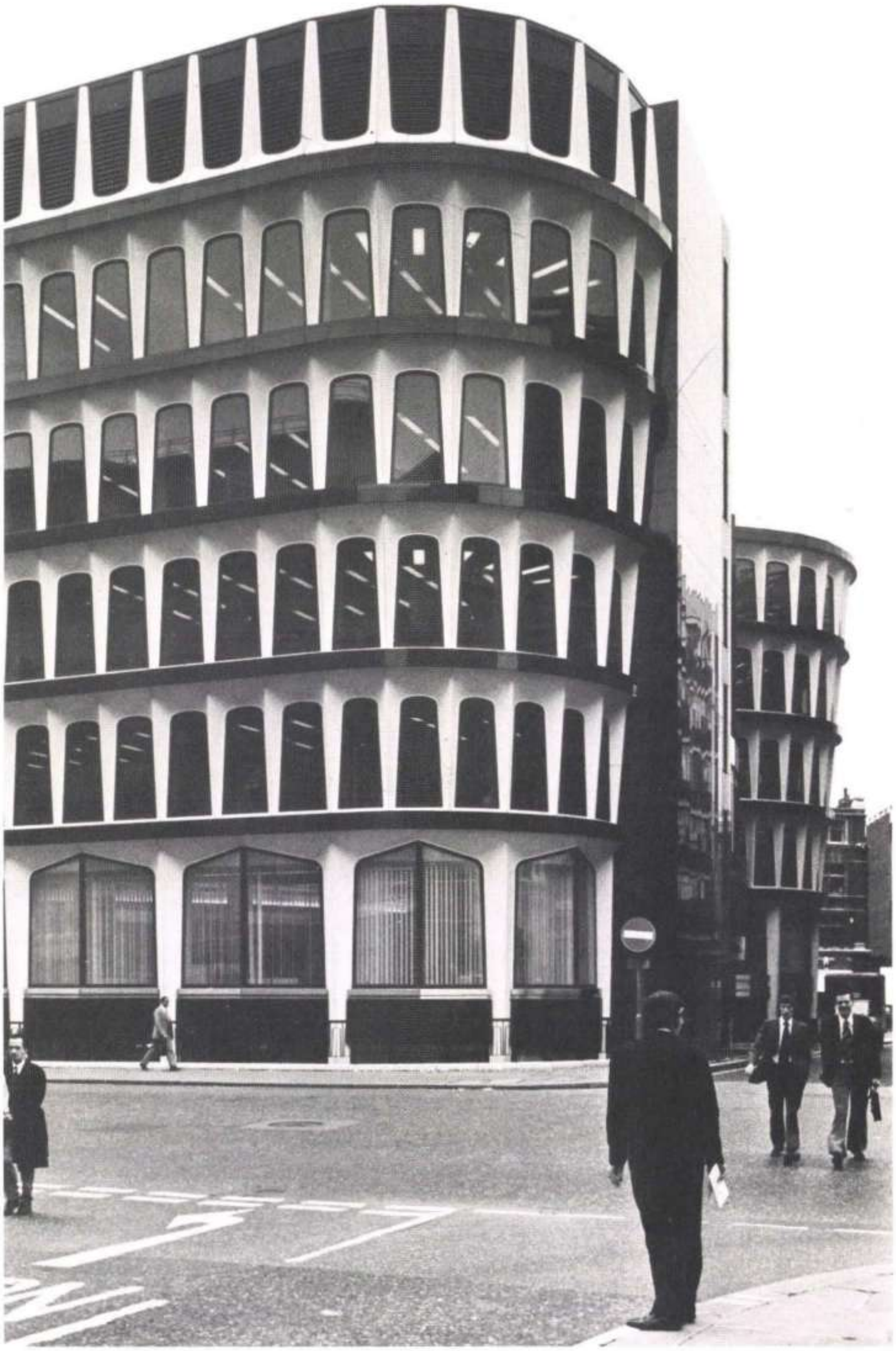
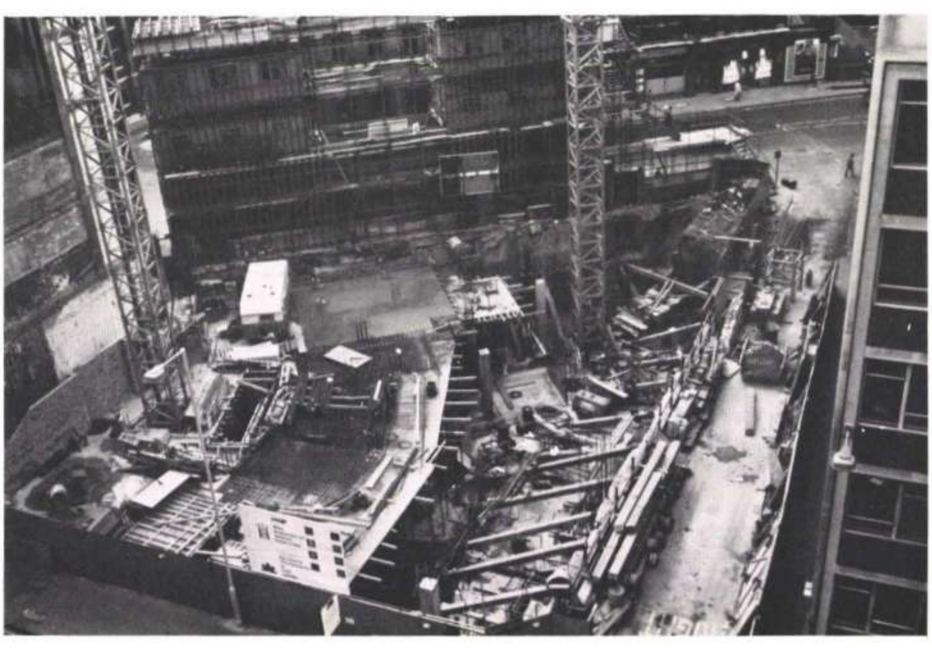


Fig. 10 right
Completed Phase I from north west, Cannon Street/Bread Street Junction (*Photo:* Harry Sowden)

Fig. 11 below
Looking out of Credit Lyonnais window towards St Paul's. GRC external cladding unit and internal finishing unit at left (*Photo:* Michael Courtney)

Fig. 12 below right
Phase I site from fire station, temporary works propping visible at right (*Photo:* Tony Dixon)



A general discussion of design possibilities showed that £1m. could be cut from the cost of the services, provided there was a services distribution zone beneath the structural slab completely free of any beams. The area of service shafts required could also be reduced and one of the basements removed. This solution was based on the air-conditioning system reverting to a general use of fan coil units.

A rapid calculation of storey heights, headroom and required zones for services and finishes showed that the structural zone was limited to 200 mm adjacent to the perimeter of the building and 250 mm in the centre. The structure was therefore determined as flat slab on columns on a raft foundation. The columns were to be on a 4.5 m square grid for the perimeter areas and on a 6 m square grid for the central areas. The façade support columns were to be inclined to avoid excessive cantilevers of the slab.

It was also felt that the vertical circulation shafts were too far from the centre of the total site development. A redesign sited the fire escape stairs on the perimeter of the building, removing the need for protected passageways, and sited the lifts at what would become the centre of the complete development, yet still be a convenient place when Phase I only had been completed. This developed into the concept of a central shaft and wing walls in reinforced concrete on the upper floors, reflecting the shape of the site, and a circular shape for the central lift shafts and service shafts at the ground floor with the reinforced concrete walls above spanning over onto columns to form a circular banking hall with the central shaft as an axis. The public pedestrian area encircled this and the banking counters of the three banks in turn formed an outer enclosing circle, with the supporting offices in the adjacent perimeter areas. All parties and authorities gave their support and consent to this unusual concept of a shared banking hall.

The total net lettable office floor area was made up by raising the ground floor 1.2 m above the pavement so that natural light could reach the perimeter of the lower ground floor which could then be used as offices.

Phasing: general

The building was conceived as a unity although it had to be constructed in two sections due to the continued presence of the fire station.

The junction between the two phases posed a serious structural question. It could be solved simply by making a complete separation between the two phases and treating them as separate buildings. This would have necessitated a double structural support line at or near the junction and did not seem to be in sympathy with the structure or appearance of the completed building. The simple structural form of the building, particularly the two-way spanning flat slab, and the raft foundations on a thick gravel layer over a considerable depth of reasonably good London Clay, did however allow the two phases to be built separately and stitched together to form one complete whole. An extensive consideration of the flexibility of the superstructure, the raft foundation and the ground geology, together with calculations of deflections and rotations, showed that elastic differential settlement would not be a problem provided the stitching of the joint was accomplished after the structure of Phase II was complete. The study also showed that the rotations from the long-term consolidation settlement would not create an unacceptable situation for the limit state of serviceability.

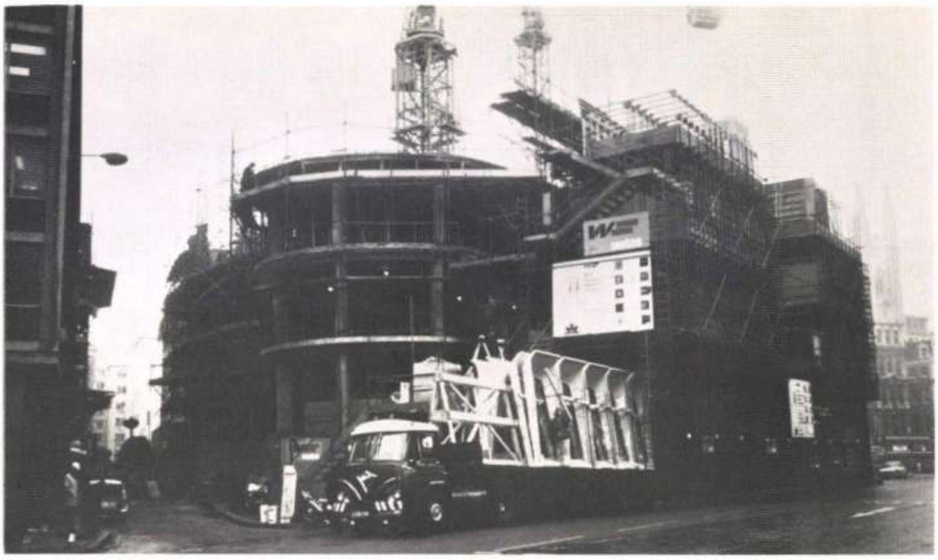


Fig. 13
Phase I from south west corner Bread Street/Queen Victoria Street Junction. Structure up to third floor, external GRC units being delivered, Wates Construction Limited gantry on the right (*Photo:* Tony Dixon)

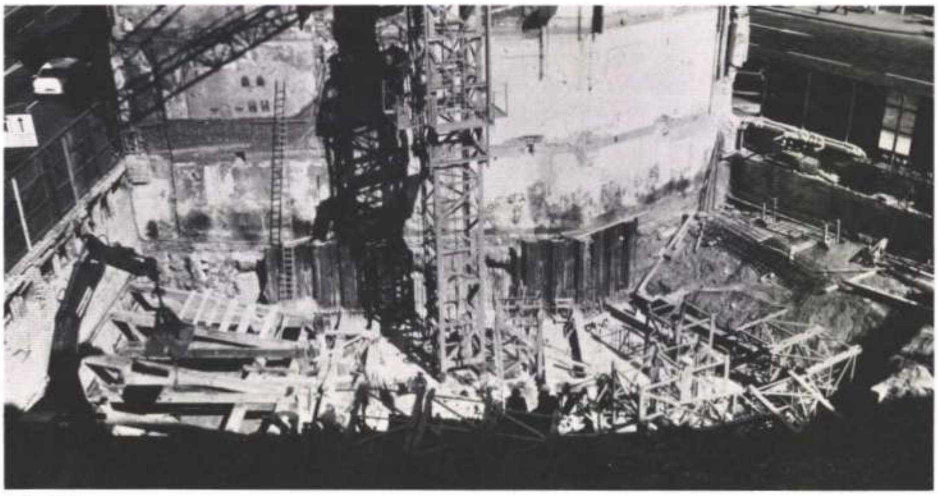


Fig. 14
Phase I site during excavation seen from the top of offices across Bread Street. The temporary propping to the perimeter underpinning is visible on the left, and the beam of retaining material to the underpinning can be seen on the right. The flatjack thrust props to the sheet piling are in the middle around the crane. (*Photo:* Tony Dixon)

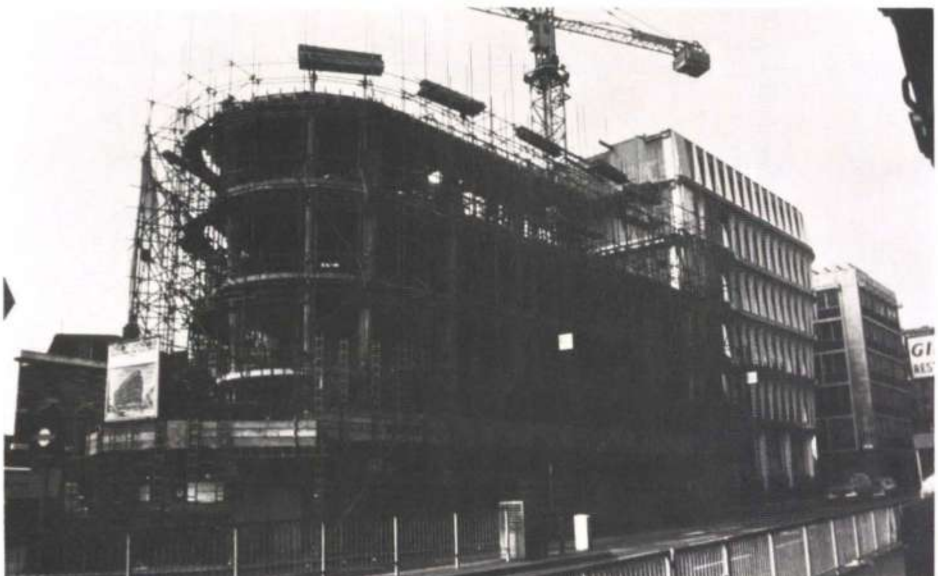


Fig. 15
The situation on site in March 1977 (*Photo:* Ray Beeney)

The architects and services engineers needed all the space that was available in Phase I so it was decided to build right up to the wall of the fire station and achieve the in situ stitch in the Phase II area. The edge of the Phase I slabs would be supported by a brick wall which would form the temporary closure to Phase I. The Phase I reinforcement would project into the Phase II site as much as possible to try to achieve tension laps but where this was not possible (and this was generally the case), the bars would project at least far enough so that lap bars could be spliced on with clamped-on sleeves such as CCL *Alpha Splices.*

Phasing: lateral stability during phased construction

The lateral stability of the building is provided by the reinforced concrete walls of the stair, services and lift shaft cores. The lateral forces are exerted by the wind and by the sloping façade columns. Compatibility of movement in regard to the latter showed that the outward tilt was mostly restrained by horizontal forces in the floor slabs, and very little by the bending stiffness of the columns. The planning of the floors did however result in these horizontal forces not being equal and opposite and the resultant force had to be resisted by the core walls.

There were three stages through which the building had to pass and in which the stress levels and distortion effects in the cores had to be considered. There was firstly Phase I with the Phase I cores; there was also Phase II which was an independent building during the construction period and then finally the combined building. The stress levels generated were not exceptionally high but twisting distortion could be a severe problem as the core walls had only small torsional stiffness.

Very early, therefore, it was apparent that one of the design constraints was that the centroid of stiffness of the cores should be made to coincide, or very nearly coincide, with the centroid of the applied forces for each of the stages. Happily this proved not too difficult to achieve even though it meant that with every change of design to Phase II, the whole stability calculations had to be revised, to show that the new and the combined cores were still adequate. This applied even when Credit Lyonnais purchased the corner site and this was added to a replanned Phase II after Phase I was complete.

Construction phase I: temporary works

Construction of Phase I started in January 1976. A major consideration for all temporary works was the short construction time and the restricted nature of the site. A prime objective was therefore to try to keep the temporary works out of the site as much as possible and particularly in the central area next to the fire station wall where the first tower crane was to be installed, on a base within the finished raft foundation.

The first operation was to tie large steel members to the fire station wall by rods anchored to the interior walls so that the existing buttresses could be removed. Secondly the whole road perimeter of the site was underpinned down to formation level and the underpinning access pits refilled. A similar operation was not possible beneath the fire station wall due to the presence of stanchion bases which were not large enough for practical underpinning operations. Many methods were considered for this temporary work, to try to keep as much of the Phase I area as possible within the finished building and to try to avoid temporary propping from this area, which would seriously obstruct the work. Eventually after advice from, and analysis by, specialists, chemical injection grouting was chosen to provide a mass gravity retaining wall beneath the fire station wall footings. The work was carried out, but an unconfined plate loading test showed that the treatment was not successful and the grouted gravel not strong enough. Part conventional underpinning and part propped sheet piling were therefore used instead; unfortunately, the congestion of the propping from thrust blocks extended the time required for construction of the superstructure by approximately two months.

Basement construction

The excavation was carried out by leaving restraining berms against the underpinned perimeter, casting the central raft area, installing props from this to the perimeter underpinning and carefully excavating the berms. Once this had been done, the rest of the raft area could be excavated and the raft concreting completed. The retaining walls could then be concreted, leaving holes round the props from the underpinning. The props could only be removed when the slab above, propping the top of the retaining walls, was cast and cured.

Superstructure: construction and completion

A superb effort by the contractor, backing in every way the hard work of the design team, ensured that Phase I was practically complete on time in April 1976 when the client moved in, as planned.

Construction: present situation, Phase II

The work on Phase II is very similar to that of Phase I. The temporary works commenced on site in April 1976 following demolition of the fire station building and work is now well advanced. The superstructure will be complete and the link-up between the structure of the phases will be made in July 1977.

Credits:

Client:
Credit Lyonnais

Architect:
Whinney, Son & Hall

Quantity Surveyor:
Wicksteed, Son & Few

Credit Lyonnais: GRC Cladding

Brian Cole

Introduction

The building stands on an important island site at the junction of Queen Victoria Street and Cannon Street in the City of London. Though limited in height to six storeys by the St. Paul's height restrictions it was considered essential that this prestige building be of striking appearance and also easily discernible from a distance.

To meet these requirements the architects proposed a highly modelled façade form, leaning out at 5° from the pavement level. The colour generally was to be off-white, with black granite bands between each storey, and the whole façade was to be drained at each level to prevent staining.

The choice of material

The original material proposed for the cladding was precast concrete but, in September 1973, Wates Property Company, as site landlords, wrote to the client expressing definite reservations about the use of a material which was not easily cleaned and which therefore, in the City atmosphere, would eventually stain. An alternative had to be found.

Enamelled pressed steel or cast iron were the first alternative materials to be considered. M. Jean Prouvé had specialized in work of this type in France, and was consulted, but preliminary investigations showed that no kiln large enough to take the mullion units for enamelling existed in this country. Other metals, stainless steel, bronze and aluminium, were considered but were rejected either due to the problem of oxidization prevention, or because they were simply too expensive.

GRP could easily be moulded to the required shape but was rejected for the normal reasons of difficulty in obtaining Class O surface spread of flame and the required fire resistance. And so the search naturally led to a relatively new material, claimed to be the non-combustible alternative to GRP, Glassfibre Reinforced Cement. The apparent qualities of GRC appeared to match exactly the list of requirements for the cladding to this building: easily mouldable; non-combustible; smooth, washable surface; off-white self-finish with little colour variation; impermeable.

What is GRC?

In early 1974 GRC was a relatively unheard-of building material. Its history of development, composition and mechanical properties are now generally well-known by those engaged in the design of buildings but they will be briefly reviewed here.

Historical development

A simple additive to cement to impart tensile and impact strength had been the dream of many researchers for some time. Fibres of ordinary E glass had been one of the many materials tried, but they proved vulnerable to alkali attack by ordinary Portland cement (OPC). E glass fibres combined with low-alkali high alumina cement was a logical progression but this suffered other problems due to 'conversion' of the cement paste. However, in 1969 a major breakthrough was made when Dr. A. J. Majumdar at the Building Research Establishment succeeded in developing a zirconium-rich, glass fibre resistant to alkali attack which could therefore be used with OPC. In the same year the BRE laid down test boards made of this new composite material, AR glass and OPC, so that the short and long-term properties of GRC could be evaluated.

The patents and rights of GRC are held by the National Research Development Corporation. Pilkington Brothers are licensed to develop the material for commercial use and have invested heavily in research and development. The BRE also continued independently to study the material characteristics of GRC.

Composition and production methods

GRC is a composite material usually comprising 5% by weight alkali-resistant, glass fibre, manufactured solely by Pilkington Brothers under the trade name *Cem-FIL*, and ordinary Portland cement to *BS12*. Generally, for commercial use, up to 30% dry sand is incorporated in the mix to act as a filler to reduce shrinkage strains.

Three important factors affect the resultant material properties of the composite, namely the water cement ratio, the orientation of the glass fibres and the density. These factors are dependent on the technique by which the GRC component is produced, of which there are three main methods: direct spray, spray dewatered and premix.

In the direct spray method a cement slurry (usually also containing fine sand) with a 0.3 water/cement ratio and a plasticizer, is sprayed onto a mould usually made of steel, timber or GRP. AR glass fibre strand is chopped into 38 mm lengths and directed into the slurry jet at the spray head. The sprayed material thus combined is compacted onto the mould, usually by means of a hand roller, and the required thickness is built up in layers to ensure adequate compaction. Spray dewatered sheets are made by a similar method but with a water/cement ratio of about 0.5. The mould, which must be flat or of only low profile, is made of a permeable material which, by applying a vacuum behind the mould, enables the excess moisture required for spraying to be removed from the GRC and achieves compaction. This reduces the final water/cement ratio to about 0.3 and thereby improves the mechanical properties of the material. Spray production methods are normally used for producing flat sheets of the

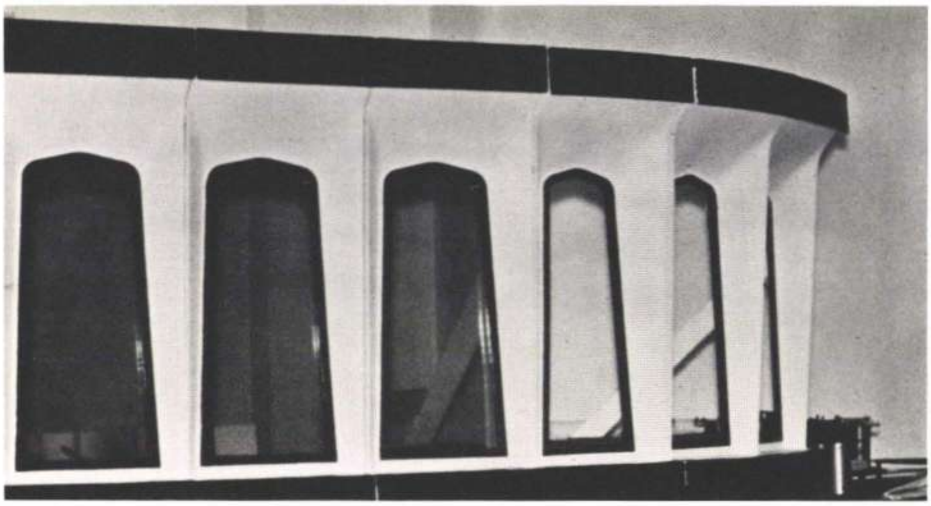


Fig. 1
Plaster model of cladding (*Photo:* Michael Courtney)

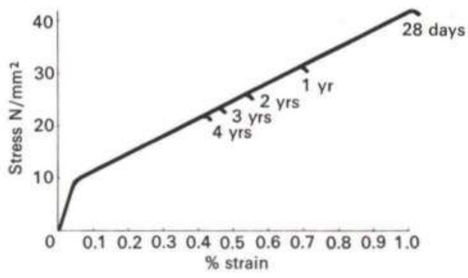


Fig. 2
Typical modulus of rupture plot for GRC

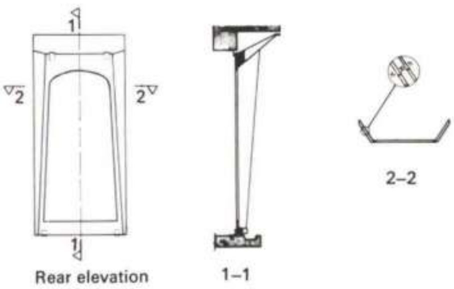


Fig. 3
Typical GRC unit

Fig. 4
Original timber pattern from which GRP moulds were made (*Photo:* Tony Read)

material, the resultant fibre orientation being a two-dimensional random array. By virtue of the cohesive nature of the material the sprayed-up sheets can be subsequently cut and folded to shape before the cement has set. In the third method of production, premix, the fibres are added to the cement before mixing. To prevent balling during mixing this method requires shorter strand lengths, about 25 mm, and the mixing process must be carefully controlled to avoid excessive abrasion of the glass fibre strands. After mixing, the GRC may be cast in open moulds and compacted by normal rolling techniques, which results in a three-dimensional random fibre orientation. Alternatively, the mix may be extruded or slip-formed which tends to align the fibres in one direction, thereby improving the mechanical properties in that direction, or it may be press-moulded, which removes excess moisture and allows immediate demoulding. Generally speaking GRC manufactured by the premix process exhibits inferior material properties to that obtained by the spray dewatered and direct spray methods of production mainly because of the variability in quality and higher water/cement ratios.

As with all cement products, hydration of the cement in GRC takes some time and it is essential that GRC is adequately cured immediately following production.

Composite properties

Much research into the composition and related mechanical properties of GRC has been carried out by both the BRE and Pilkington Brothers. The ultimate tensile strength (UTS), bend-over point (BOP), strain-to-failure, fatigue life and Young's modulus have been investigated by means of direct tension tests. Bending tests have been used to establish the modulus of rupture (MOR), limit of proportionality (LOP), creep strain and stress rupture behaviour. The shear and impact strengths, thermal and permeability properties and the performance in fire have also all been thoroughly investigated.

To establish the durability of GRC many tests have been carried out on specimens of the material which have been stored for periods of one to five years under the following conditions: dry air; water immersion and natural weathering. The direct tension test proved very difficult to carry out accurately on aged GRC due to its brittleness, so the four point bend test has been round to give more consistent results. In this test the central portion of a small flat plate of the material, usually 150 mm long by 50 mm and known in GRC technology as a coupon, is subjected to pure bending stress and the resultant deflections are relatively large and are therefore easily measured.

In the four point bend test on fresh GRC the material behaves elastically up to the LOP, full recovery is obtained on removal of the load, and simple bending theory provides the tensile stress/strain relationship. In this linear region the fibres within the mix help suppress the propagation of cracks due to flaws in the matrix and thus allow the elastic region of the cement to be fully utilized. The LOP, though essentially a matrix-dependent property, is consequently higher than for plain cement and is usually within the range of 14–17 N/mm² for fresh spray dewatered material. In addition the fibres increase the stiffness of the matrix and give a Young's modulus of 15–25 kN/mm², the actual value depending mainly on the composite density.

The modulus of rupture is obtained from the four point bend test by dividing the ultimate bending moment by the elastic section modulus. Typical 28-day MOR values for spray dewatered material are within the range 35–50 N/mm² but, because of the redistribution of stresses which takes place in the tension zone of the material, with a consequent shift in the position of the neutral axis, the MOR is an apparent stress rather than a true stress value. The behaviour of the material in the ductile region between LOP and MOR becomes essentially fibre-dependent as the tension force is gradually transferred from the matrix to the glass fibres. In this region the fibres induce fine, well-distributed cracks throughout the matrix, unlike the gross defects associated with reinforced concrete at yielding of the steel.

In spite of the difficulties experienced in obtaining the direct tension values some carefully controlled tests have been carried out in order to correlate the material properties obtained from the direct tension and the bending tests. These tests have shown the bend over point (BOP) to be 70% of the corresponding value, the LOP, obtained from the bending test, and in addition that this ratio increases in the ductile region until at rupture the MOR/UTS ratio is approximately 2.5.

The compressive stiffness of GRC is similar to that in tension up to the LOP but continues in an elastic manner almost up to the failure load, about five times tension value. In the case of compression the fibres act to resist the associated shearing and splitting forces.

The resistance to shear of GRC is largely dependent on the orientation of the fibres and varies between about 2 N/mm² where the fibres are inoperative, up to about 27 N/mm² where they are arranged so as to be fully utilized to resist the applied shearing force.

As the high local stresses are distributed throughout the matrix by the glass fibres, GRC possesses a high resistance to impact, with strengths in the region of 20–30 Nmm/mm².

Durability

The material values quoted above are generally 28-day strength properties. The durability tests have shown that significant changes occur in the mechanical properties with age and exposure conditions. Obviously with a material invented as recently as 1969, determination of the long-term properties is difficult and requires extrapolation from short-term test results. This is an extremely difficult problem as, while it is known that AR glass is alkali-resistent, the exact mechanism of the change with time of the composite properties is not fully understood and is further complicated by the difficulties in obtaining the direct tension strengths of a material which becomes brittle. For some time there was some disagreement between Pilkington Brothers and the BRE on the matter of extrapolating the long-term strengths. Pilkingtons maintained that an exponential time/strength decay curve best fits the short-term test results. They felt that the material degradation eventually ceased with time and that the 20 year value estimated by curve fitting was asymptotic to a stable value. However, the BRE suggested that extrapolation on a log-time base, by which the decay curve is heavily influenced by changes in early years which are assumed to continue at a reducing rate until a cut-off point, 'the matrix level', is reached, was the correct method of determining the long-term material properties.

With the publication in 1976 of the *BRE Current Paper CP38/76* this problem has been somewhat resolved. This document gives the long-term properties for GRC according to an exponential curve, and an alternative set derived from a logarithmic decay curve, and the final judgement is left to the designer.

Eight years have now elapsed since the invention of the material and another set of points on the strength/time curve would obviously help a great deal in determining the correct curve to use, but unfortunately the BRE is now running short of those original 1969 specimens and is anxious to save some for the 10 year results!

Our role in the cladding design

Having selected GRC as the most suitable material for the cladding, the architect asked us to provide him with a design service. This service was to include the following: general advice on GRC cladding panels; outline scheme and structural design including fixing details for each specific panel type; assistance in obtaining approval from the district surveyor for use of this new material; close liaison with the manufacturer to establish final panel details; assistance in preparation of a GRC specification and assistance with quality control.

Our involvement in this project has been described as somewhere between precast concrete panel design, for which we normally take full design responsibility, and the use of a proprietary lightweight curtain walling system for which the manufacturer would be responsible for the design and detailing and our involvement would be only marginal. In fact our design involvement was very much closer to the former than the latter in this case.

Structural design of the cladding

At the time of the decision to use GRC for the cladding material, early 1974, there was sufficient information available within the R & D section to enable us to provide a very preliminary scheme structural design for a typical cladding unit. The panels are non-load-bearing but must span between the concrete floors against wind loads. We worked within the constraints established by the architect at that time, the required external profile, the inter-relationship with the load-bearing structure (the sloping façade columns are wholly within the cladding zone), the tolerances required between individual units, the waterproofing and drainage requirements and the implications of the design on the fixing details.

The external panel which evolved comprised two hollow box section legs spanning between the floors in simple bending, a top section cantilevered out beneath the projecting floor above, and a hollow base section, all forming a complete frame to contain the double glazed window unit. A major advantage of GRC is that with no steel reinforcement to protect from corrosion, coupled with the fact that the material is very cohesive, very thin material thicknesses are possible. We envisaged the direct spray method of production and proposed a GRC thickness of 10 mm for the units but cautiously reserved the right to amend this as we found out more about acceptable long-term stress values for the final design.

Fig. 5
Spraying-up process
(*Photo:* Brian Cole)

Fig. 6
Compacting GRC with hand roller
(*Photo:* Brian Cole)

Fig. 7
Trowelling-off
(*Photo:* Brian Cole)

Fig. 8
The curing enclosure
(*Photo:* Michael Courtney)

The fixing details we proposed for the typical panels were similar to those commonly used for precast concrete panels: two stainless steel angles bolted to the slab and the beam soffit, with stainless steel bolts threaded into sockets cast into the rear of the panel. Positional tolerance was obtained by slotted holes in the angles and packing washers in the direction in which positive restraint was required. It was essential that no vertical load was imparted by the structure to the panels so vertical slotted holes were provided in the top fixing angle with *Teflon* washers to allow for relative vertical movement of the structural frame.

To prevent misuse of the material all manufacturers of components in GRC are licensed by Pilkington Brothers. One such licensee, Portcrete Ltd. of the Bath and Portland Stone Group, was nominated as sub contractor for the supply and fixing of the cladding in the summer of 1974 and entered into the project with much enthusiasm. Our early discussions with Portcrete Ltd. confirmed the feasibility of our earlier scheme work and enabled us to proceed further with the structural design.

As no codes of practice existed for design in GRC we decided that a limit state approach, taken from first principles, would give us a better understanding of the finished product. Three basic parameters had to be established:

(a) The design load
(b) The mechanical properties of the material
(c) Appropriate factors of safety.

The applied loading on the cladding panels in their final condition was confined to wind loading and we were able to directly apply the cladding pressures obtained from *CP3: Chapter V: Part 2.* A consideration of the building form showed that there were unlikely to be any localized high pressure areas.

The serviceability limit state was easily justified. Using the well-established value for the Young's modulus of the matrix we soon found that the lateral deflection and associated cracking of the unit legs, even under maximum wind loading, would be negligible. The critical limit state was therefore that of collapse but the appropriate material values to be used for this condition proved to be far more difficult to establish. The important material property for this condition in the bending of a box section is the long-term ultimate tensile strength. The nearest property to this for which extensive test data were available was the modulus of rupture obtained from a four-point bend test on a flat coupon. This information therefore related to a cross-section different from the hollow box sections proposed for the unit legs, was for a different loading condition and, in addition, also dealt only with short-term test results. Both the BRE and Pilkington Brothers were certain that the MOR value decays with time but neither knew for certain to what extent. After much deliberation we therefore decided to base our design on a specified minimum LOP as this is essentially matrix-dependent and therefore less vulnerable to the degradation of composite properties.

Our assessment of the factors of safety to be used in the design was based on papers on the subject written for the BRE, the Construction Industry Research and Information Association (CIRIA) and the Institution of Structural Engineers. A list of relevant parameters was carefully drawn up and appropriate partial factors of safety were allocated to each as follows:

Loading	1.4
Accuracy of analysis including material strength	1.3
Workmanship	1.1
Mode of collapse	1.1
Consequence of failure	0.9

These factors, when combined, give an overall 'global' factor of safety for the design of approximately 2.0.

We therefore now had established our basic parameters for design – load, material strength and factors of safety, and we were able to submit these to the local authority for approval. During the development of the design approach we had many constructive meetings with the District Surveyor who was therefore fully conversant with our proposals and he readily gave his approval to the use of GRC for the cladding material.

Specification

Together with Arups' R & D group, the architect and the QS, we developed a detailed performance specification for GRC soon after the basic decision to use the material had been made.

When Portcrete Ltd. were nominated as supply and fix sub-contractors we were able to involve them in the development of the specification to cover materials, workmanship, manufacture and quality control. Their involvement proved invaluable in that they were able to carry out tests to confirm that not only could the design team get what it wanted in terms of strength and quality, but also that the required material properties were obtainable under normal manufacturing conditions by careful, but not excessive, quality control and testing.

The design for the GRC mix was included in the materials section by stating limits for the mix proportions and allowing the manufacturer to propose a specific mix to fulfil the performance requirements. To prove that the proposed mix, equipment, personnel and technique would produce the required properties, a series of preliminary tests and samples were required.

For routine quality control during production we decided to use the well-established four-point bend test, carried out on coupons cut from test boards sprayed up in step with the first and last panels of each day's production. Although the LOP stress was used in the design it was generally agreed that the MOR is the simplest and most consistent value obtained from this test and it was therefore decided to use this value for specifying and checking the strength of the production GRC. The approach to specifying the MOR was similar to that used in *CP110* in that a characteristic strength was specified, above which a certain acceptable percentage of quality control tests must fall, and by stating a lower limit below which no single result may fall. Allowing 1 in 20 results to fall below the characteristic strength, the target mean strength to be aimed at by the manufacturer is:

$M_{TM} = M_k + 1.64 \times$ standard deviation.

The characteristic MOR (Mk) required in our specification was 23 N/mm² at 28 days. The preliminary tests were designed to ascertain a realistic standard deviation and to enable us to establish a relationship between the MOR and the LOP used in the design.

An important aspect of the specification is the requirements incorporated within it to reduce the possibility of surface crazing of the material, a problem which had beset previous projects in GRC. Discussions between Pilkington Brothers and R & D had shown that surface crazing could be significantly reduced in three ways: by elimination of thick 'gel' layers of unreinforced matrix at the surface, by using up to 30% of a particular sand filler and by controlling the curing process. These three factors were incorporated into the specification as follows: the unreinforced mist coat sprayed on the mould before spray-up of the composite was limited to 0.5 mm, the mix proportions were set within limits and included a fine sand filler, and a strict curing specification (10 days in a curing enclosure at 95% RH) was included in the section on manufacture.

Tests

In addition to the tests to be carried out under the specification we felt that, to economically and safely design the units, it would be useful to carry out a further series of tests on a structural section similar to that proposed for the spanning mullions of the units. The purpose of these tests could be summarized as follows:

(1) To determine the relationship between the properties of the flat rectangular section of the quality control coupon and the rectangular hollow box section

(2) To determine the efficiency of the GRC spray-up construction joints between the outer flange and the short webs of the box section

(3) To obtain further information about the shear and torsion characteristics of GRC.

(4) Tests were also required to justify the design of the proposed cast-in fixings, both for pull-out in the working condition and shear in lifting.

Pilkington Brothers agreed to carry out these tests in their laboratories on a series of box sections, made specifically for the purpose by Portcrete Ltd., of proportions similar to the mid-height cross-section of the mullions of a typical cladding unit, and also a series of tests on some cast-in sockets of the type proposed for the cladding fixing. The box sections were constructed in a manner identical to that used for the legs of the unit: the outer flange is sprayed up first against the mould, a non-structural 3 mm thick pre-formed GRC former of inverted U section is placed on this, and the spray-up is continued over this former. The joint between the flange and side walls is effected by rolling projecting strands from the lower portion into the new spray-up, forming the side legs.

The results of this series of tests were most useful in that they confirmed our basic design approach, they provided the relationship between coupon and box-section behaviour which we could incorporate directly into our design calculations and they proved the spray-up construction joint to be entirely efficient. The fixing tests confirmed that the proposed fixings would be entirely satisfactory in both the tension and shear conditions.

Later, during the production of the units, Pilkingtons were provided with the opportunity to test a full-size unit to destruction. The unit was mounted horizontally, supported at the fixing position, and loaded by means of bricks in a manner to represent wind suction. This test was very useful in that it provided us with information on the overall behaviour of a complete panel. The unit proved to be both strong and very stiff, the load at LOP being approximately 3.3 times the design wind load, and the associated deflection was only 1.1 mm. The panel, which, it should be remembered, exceeded the specified minimum thickness and material strength requirements, sustained a maximum load of nearly nine times the design wind load for 42 hours without either material failure or excessive deflection.

Panel production

Immediately following receipt of the design geometry from the architect, Portcrete Ltd. produced a full-size timber mock-up of a typical unit. The architects were able to inspect this mock-up and make one or two minor geometrical changes. The timber unit was altered accordingly and four GRP moulds were produced from it. GRP is an excellent material for mould manufacture as it produces a dense, smooth, cast GRC surface. The first panel was produced in December 1974 and delivered to site in February 1975. Production of 400 panels for Phase I took approximately eight months and the last Phase I panels were delivered to site in September 1975. The maximum number of panels produced in one day during this period was six.

Of the 400 panels produced, about six were rejected at the factory. This was due either to problems with curing or dimensional inaccuracy outside the tolerance range. In the design of the mould, Portcrete Ltd., using their experience gained with sprayed-up flat sheets of GRC, had allowed 1 mm/metre for shrinkage. In the event the initial shrinkage proved much less than this, resulting in some oversize panels and Portcrete Ltd. subsesequently reduced the mould size.

Erection

The use of GRC provided a panel far lighter than a similar sized panel in precast concrete. This aided the transportation and erection of the panels; hoisting was simpler and cheaper and storage problems were less. Care was needed during transportation but generally more units could be carried in each journey. In addition, the ground floor units could be made in far larger sections than could have been handled in precast concrete and the need for handling reinforcement was obviated.

The erection of the units was straightforward and proceeded with only minor problems due to the concrete support structure being, in one or two instances, outside the tolerance limits. However, during the course of the erection, the BRE issued their conclusions on long-term strengths of the material in the form of CP38/76 mentioned earlier. The minimum possible strengths quoted in this document were slightly less than those used in our design and a check on the design calculations

Fig. 9
Load test on a full-size panel (*Photo:* Pilkington Brothers)

Fig. 10
Internal view of south-west corner of Phase 1 (*Photo:* Michael Courtney)

Fig. 11 below
External view of south-west corner of Phase 1 (*Photo:* Harry Sowden)

Fig. 12 right
GRC cladding panel for the ground floor of Phase 2 being lifted from lorry (*Photo:* Ray Beeney)

showed that some of our panels would need strengthening to avoid exceeding the absolute minimum strengths stated. We discussed this information with the architect and we agreed that, whatever the likely interpretation of this new strength data, it was undesirable to take a risk that further strengthening might be required some time in the future. This was not an engineering appraisal but an assessment of the client's best interests. Additional restraint to the critical units was provided by bracing the units by a steelwork member located in between the external and internal cladding units.

Conclusions

GRC has undoubtedly provided very distinctive cladding on this ambitious project. The design team have been extremely pleased with the performance of both the material itself and the organizations involved in its production: Pilkington Brothers, who developed and supplied the AR glass fibre and much of the research required for the initial design, and Portcrete Ltd. who manufactured the units to a consistently high standard on programme, and who also readily produced many of the samples and tests required during the early development of the design and specification. Portcrete were very much assisted in their work by the skills, organization and enthusiastic co-operation of the main contractor, Wates Construction Ltd.

The light weight of the units made possible by using GRC contributed not only in direct ways to the successful completion of the building on programme, but also in indirect ways by reducing the structural support required and foundation sizes. It is also doubtful that the units could have been economically produced, in the time available, in any other material.

It is accepted that the stresses used in the design are very low compared with the short term data available for the material. The material is not yet suitable for use for major structural elements which must retain their strength for a long period. Better use of its strength properties can only be made in short-term situations – shuttering, permanent formwork and replaceable components. However, many of the other important qualities of GRC: its mouldability, surface finish and light weight, were successfully utilized to the full on this project.

Centre Pompidou

A Special Award of The Institution of Structural Engineers has been made to Ove Arup & Partners for 'their contribution to the creation of the Centre Nationale d'Art et de Culture Georges Pompidou, Paris.' (Architects: Piano & Rogers.)

Only eight of these Special Awards have been made in the last 10 years and the firm is the first to receive a second one. Arups gained their first in 1973 for their work on the Sydney Opera House.

Fig. 1
View of front façade from the piazza
(Photo: Poul Beckmann)

Fig. 2
View from Rue St. Marie showing services on rear façade at right of photograph
(Photo: Martin Charles; courtesy of *Architects' Journal*)

Fig. 3
The Forum: ground floor entrance
(Photo: Martin Charles; courtesy of *Architects' Journal*)

Fig. 4
View of the Library
(Photo: Bernard Vincent)

Fig. 5
General view
(Photo: Bernard Vincent)

Fig. 6
Corner view from the piazza
(Photo: Poul Beckmann)

Fig. 7
The façade showing escalators and tubular galleries
(Photo: Bernard Vincent)

Fig. 8
Interior of escalator
(Photo: Bernard Vincent)

Fig. 9
Museum of Modern Art
(Photo: Martin Charles; courtesy of *Architects' Journal*)

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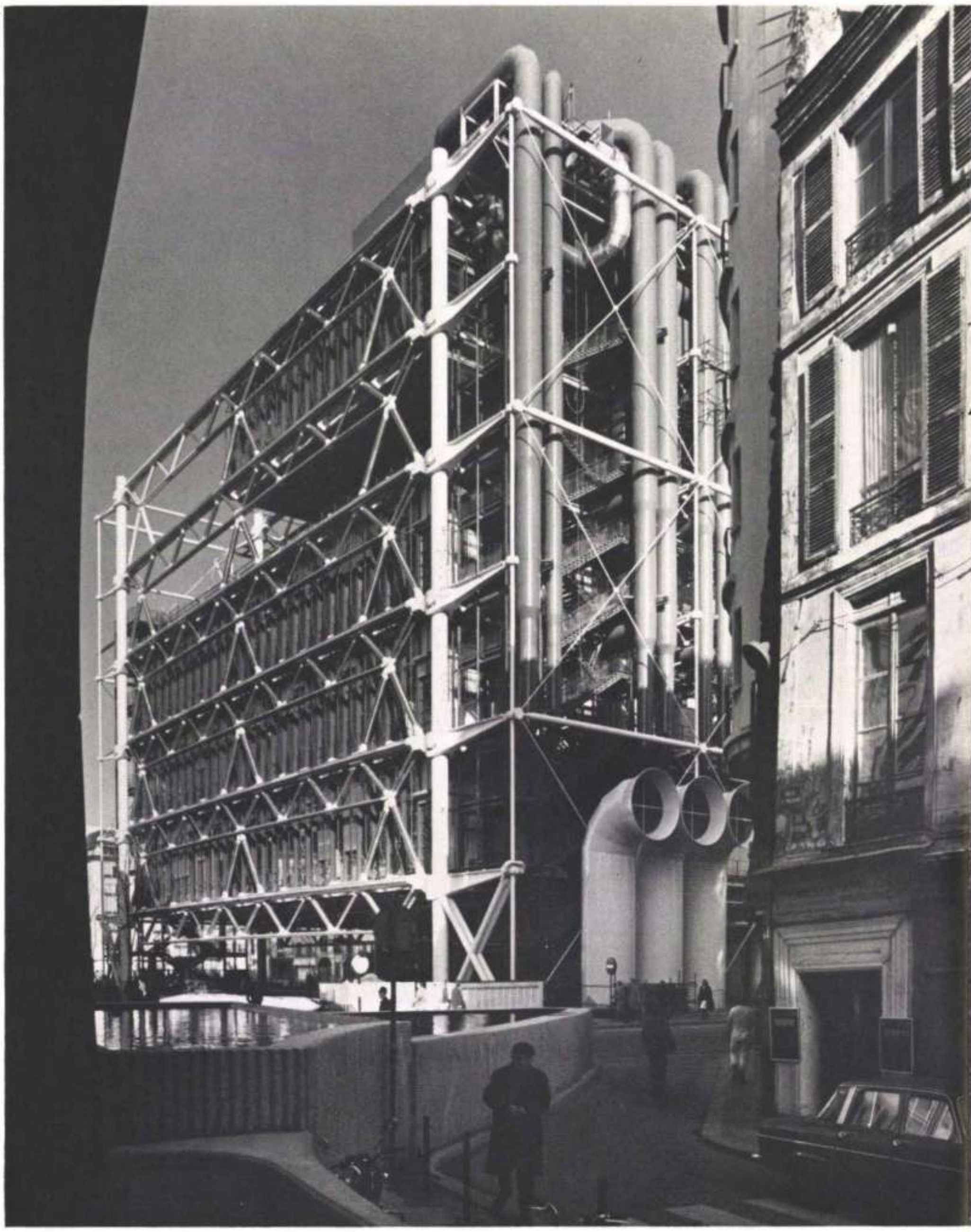

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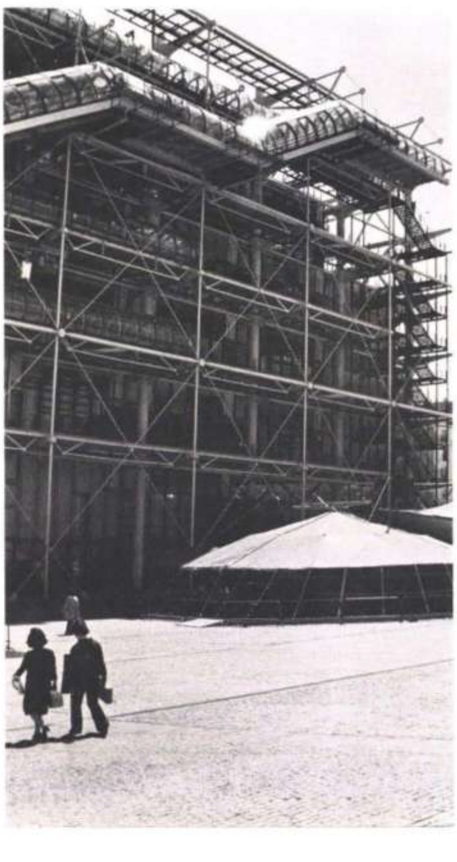

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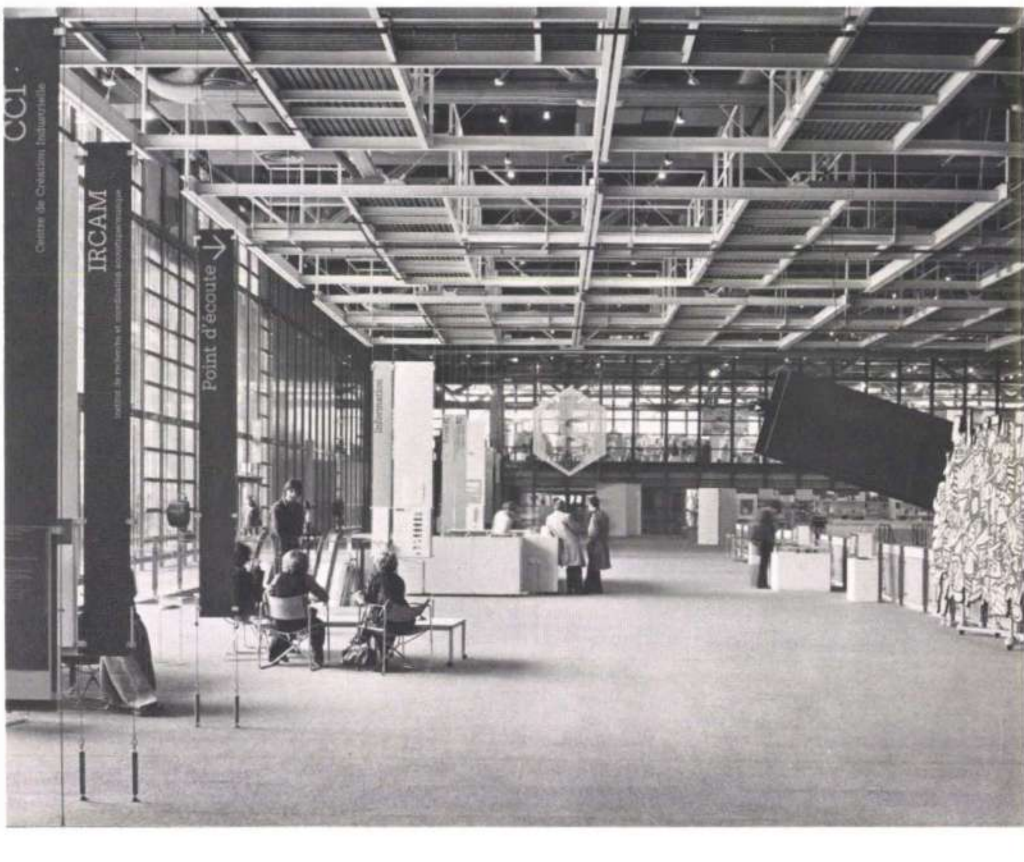

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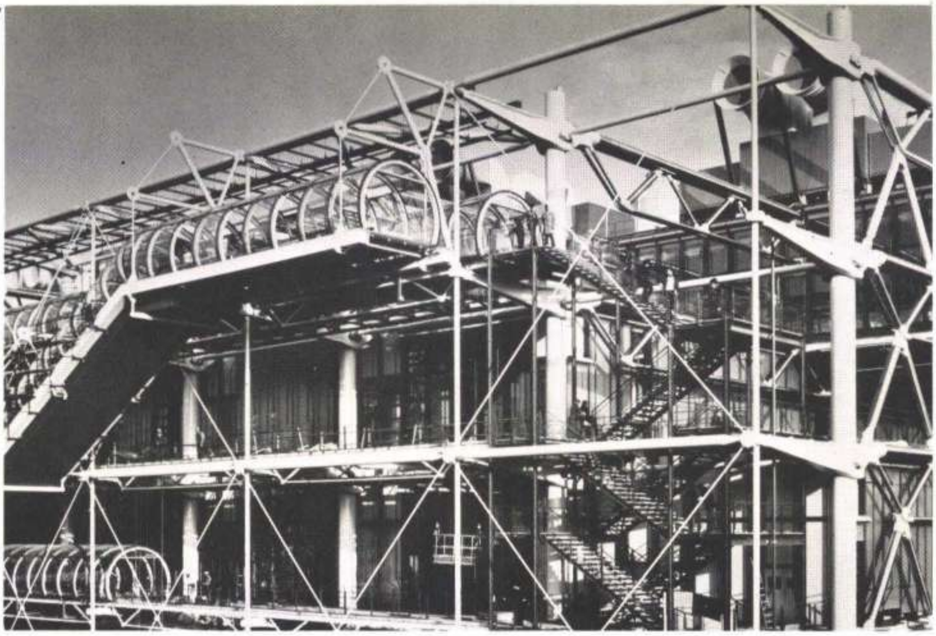

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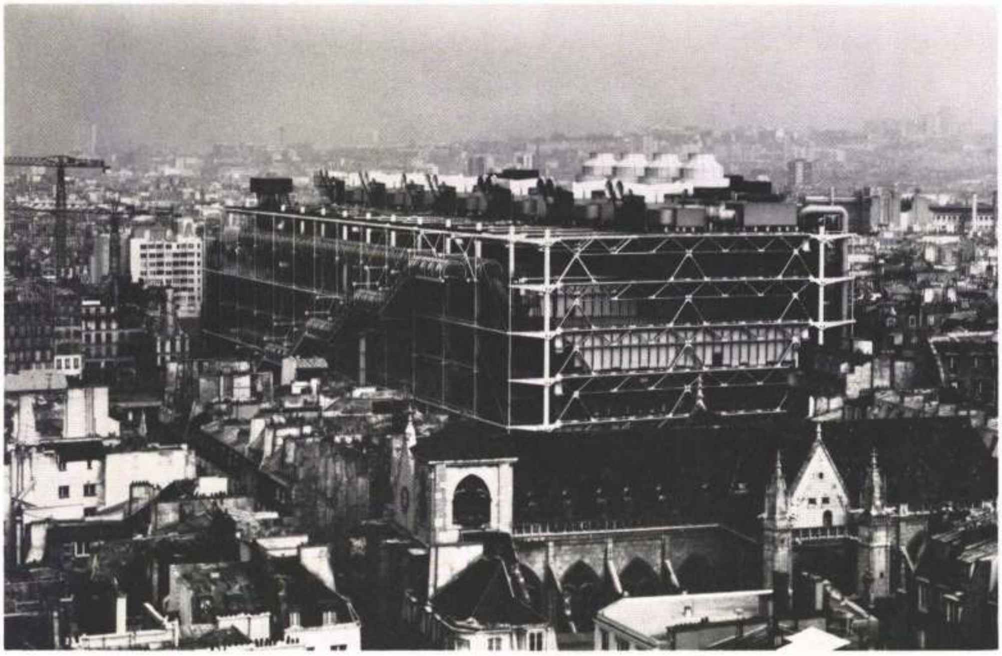

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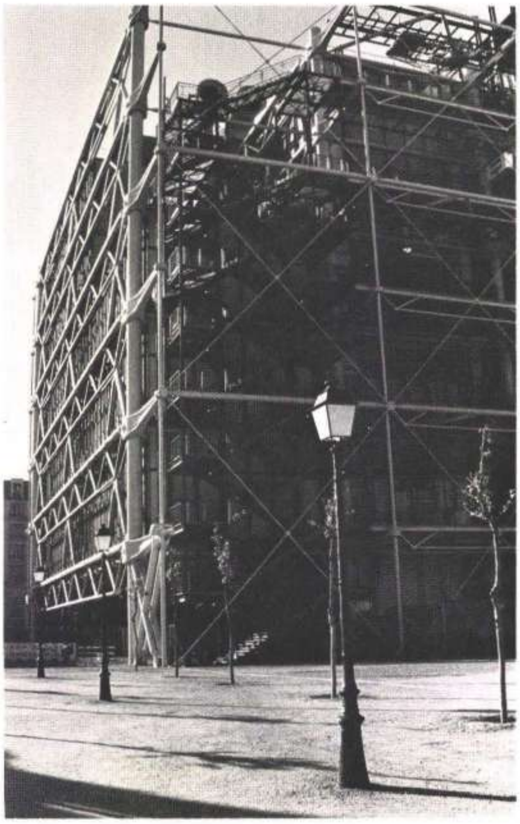

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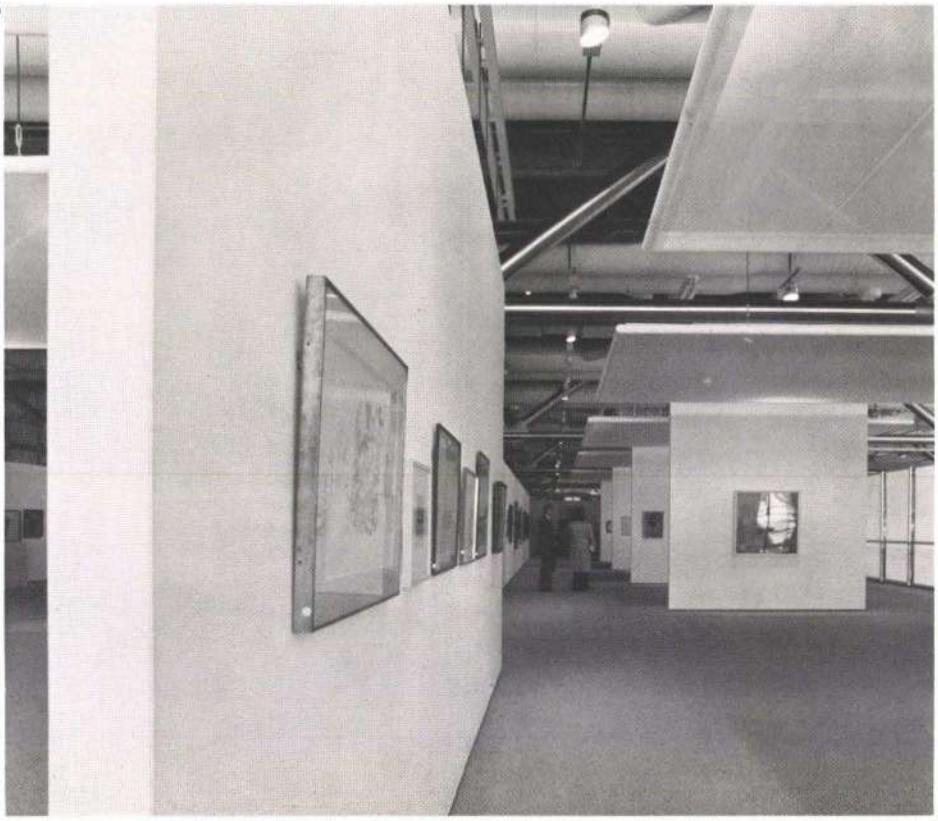

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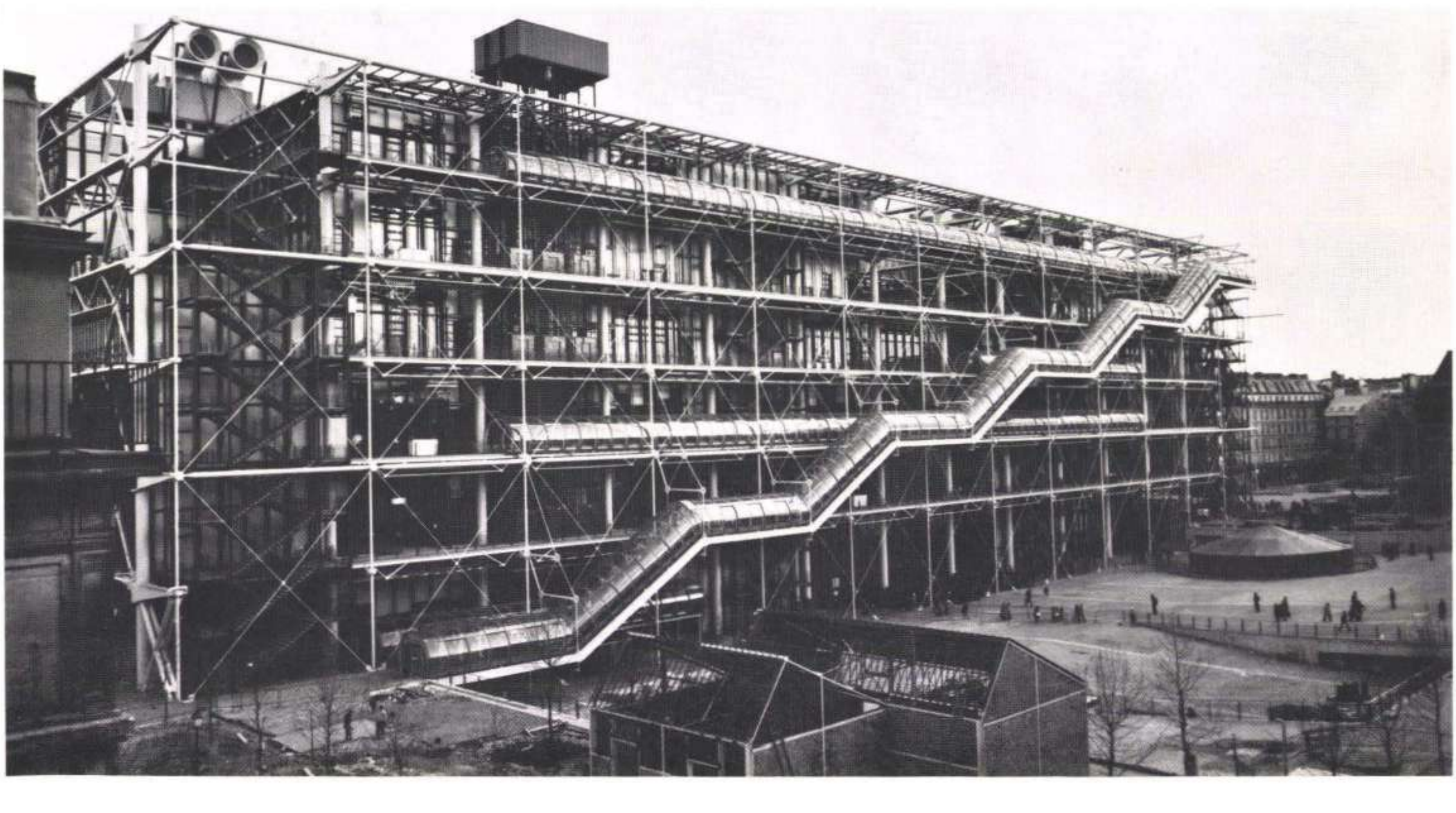

Fig. 10
Fifth floor exhibition area
(Photo: Martin Charles; courtesy of *Architects' Journal*)

Fig. 11
An escalator and tubular gallery viewed from the interior of the building
(Photo: Bernard Vincent)

Fig. 12
The front façade and piazza
(Photo: Bernard Vincent)

Fig. 13
Escalator and walkways
(Photo: Martin Charles; courtesy of *Architects' Journal*)

Fig. 14
View of rear of building in Rue St. Martin showing services
(Photo: Bernard Vincent)

Fig. 15
Close-up of the escalators and tubular galleries
(Photo: Bernard Vincent)

Fig. 16
The Forum
(Photo: Bernard Vincent)

Fig. 17
Exhibition area
(Photo: Bernard Vincent)

Figs. 18 and 19
Services
(Photos: Bernard Vincent)

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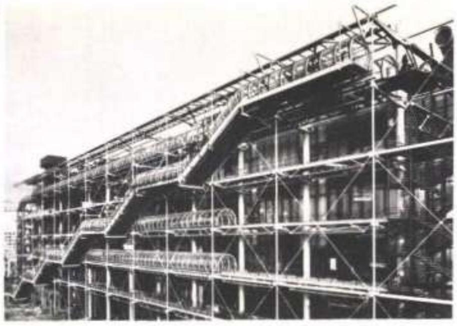

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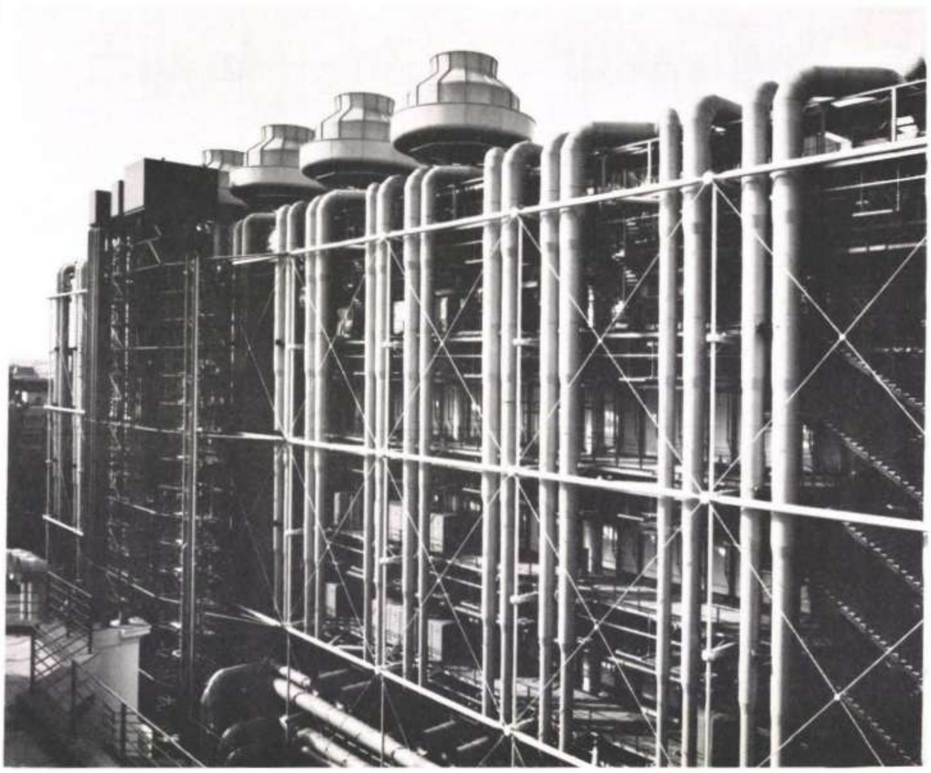

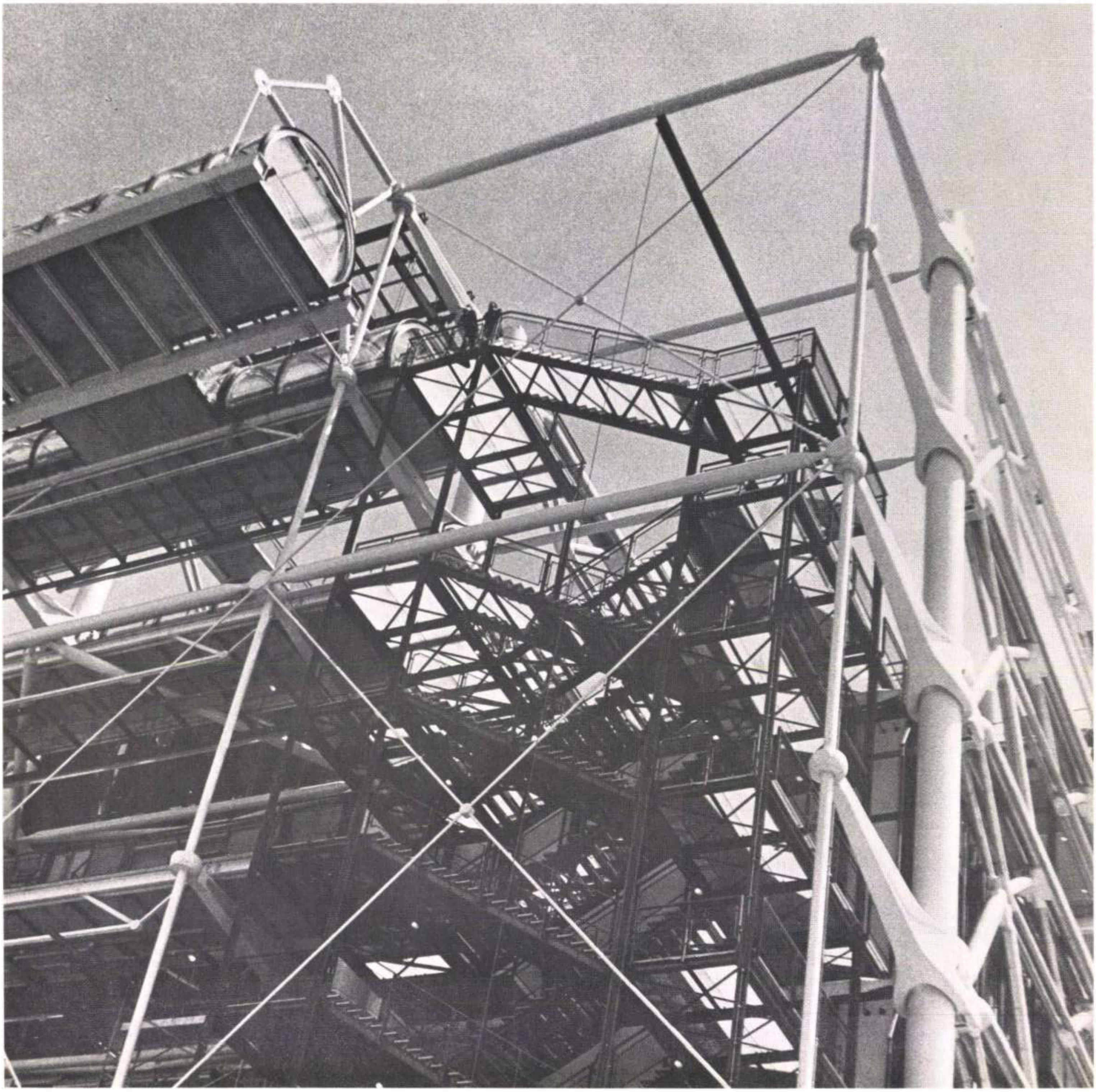

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