THE ARUP JOURNAL

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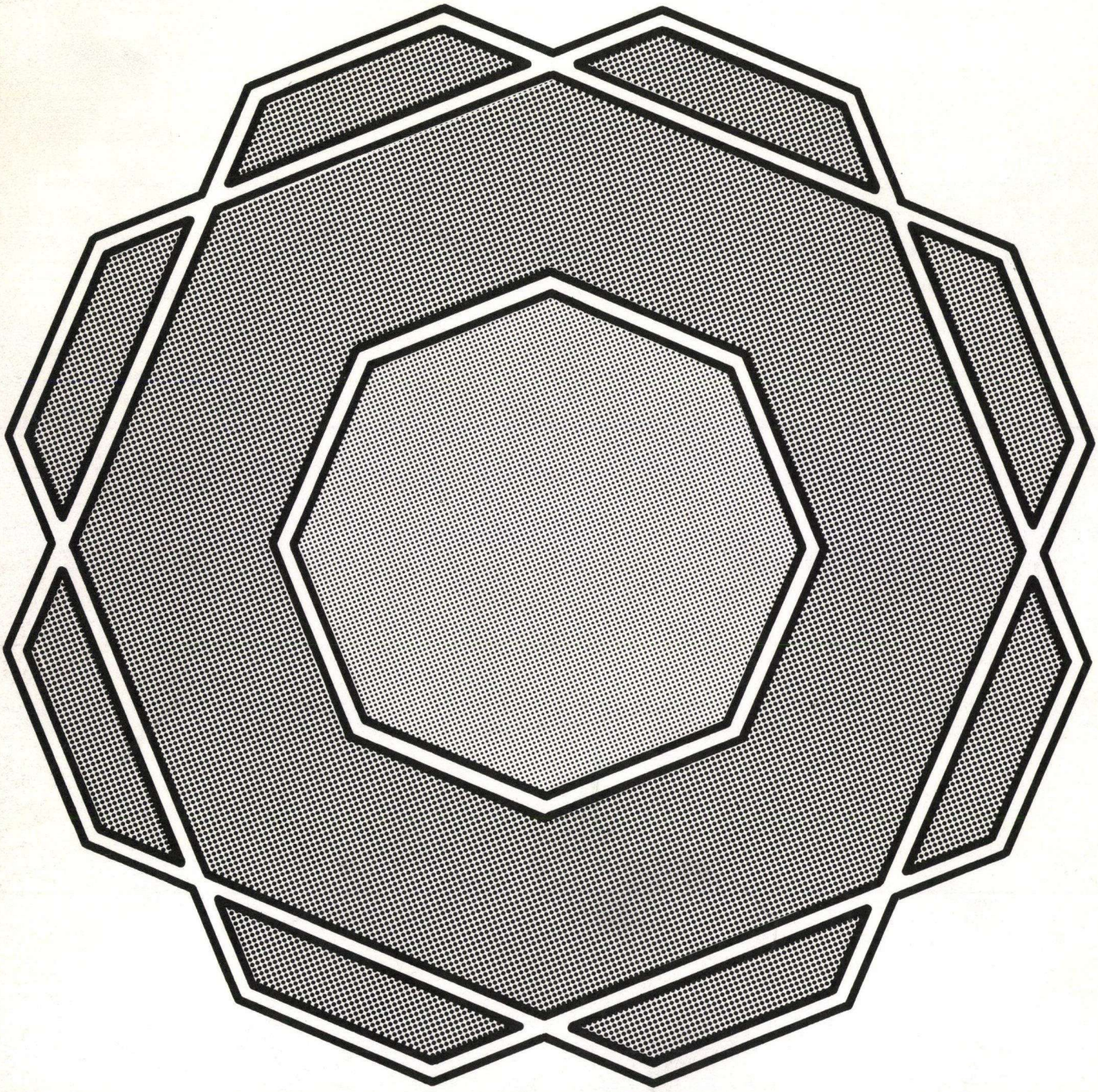

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Front cover: Plan of Island Site, Addiscombe Road. (Drawn by: Susan Pickard)
Back cover: Sheffield University Concourse central support. (Photo: Henk Snoek)

The Island Site, Addiscombe Road, Croydon, Surrey

Tom Henry and Peter Ryalls

General description

The development is situated on a triangular-shaped traffic island, (Fig. 1) bounded on all three sides by a one-way, high density traffic system. There is a road junction at each corner of the triangle. It is connected by three pedestrian subways to both the railway station and adjacent office developments, thus eliminating the need for pedestrians to cross the roads at ground level.

The site is extremely congested. This is caused partly by the circular car park excavation being tangential to all three roads, but is increased by a substantial private house which stands at one end of the site. This house was to have been demolished. It is, however, still there and has every likelihood of remaining there for several more years. It is tenanted, on one floor only, by a lady solicitor who does not wish to leave the house and has so far maintained her right to remain there herself, and to ensure that the building also remains until her lease expires several years hence. It is thus necessary to build the tower block and leave the house and its occupant undisturbed.

Due to re-alignment of the road system, part of the old road is incorporated within the car park area, and although it would never be used again as a road, until a court order could be obtained for a permanent road closure, the law held that it should be possible to re-instate the road if required. Construction started whilst contending parties were locked in legal battle. Rearguard actions against the closure were fought by the opposition, and construction was fast approaching the point where it would not have been possible to reinstate the road and still proceed with the development without some revision to the scheme, when victory was achieved and construction continued, thankfully without a halt. A developer's life is not always an easy one!

The architects, R. Seifert and Partners, had been asked to provide a 23-storey prestige office block of distinctive appearance, together with the maximum possible car parking storage below ground.

Maximum car parking space has been achieved by inscribing, within the triangle, a circle of 180 ft. (55 m) diameter (formed as a 20 in. (510 mm) thick concrete diaphragm wall) which is tangential to all three sides. The centre of the circle, which is useless for car parking, is occupied by an octagon (45 ft. (13.7 m) across the flats). The octagon is the concrete walled service core of the building above and contains six passenger lifts with a speed of 700 ft./min. (213 m/min.), toilets, two staircases, and sundry vertical ducts for all the mechanical and electrical services.

The car park consists of three turns of a screw. The helix serves both as park and access to the lower levels.

Cars park radially against both the perimeter and service core walls, leaving the space between to act as the access road. There are places for 180 cars.

Above ground, the building is unusual in plan (Fig. 2), the perimeter being a twenty-four sided figure formed by superimposing two squares above each other, one square rotated at 45° with respect to the other, and then cutting the corners off each square to form two unequal sided octagons superimposed on each other. (In the initial schemes the corners were not cut off.)

At each floor level the first of these two octagons forms the office floor, and the projecting parts of the second octagon, not covered by the first, form four balconies. On the floors immediately above and below the floor being considered, the second octagon forms the office space and the projecting parts of the first, form the four balconies.

The effect of rotating the floors and providing balconies is to produce a building which externally gives the impression of being circular with bold projecting facets, rather like a very knobbly pineapple (Fig. 3), but which has the great advantage of having office floor plans which are almost square. The balconies provide the further advantage of shielding the windows by an 11 ft. (3.4 m) overhang and also serve to collect rainwater at each level and thus eliminate the problem of progressively larger quantities of rainwater passing down the face of the building to be collected somehow before ground level is reached. This problem is not always dealt with adequately at the design stage.

Between ground and first floors the architects have boldly expressed the vertical structure, a characteristic for which they are well known (Fig. 4). The glazing line is withdrawn inside the line of eight perimeter columns, the plan shape varies, and, in radial elevation, the columns slope slightly towards the centre of the building between lower ground floor and ground floor and then sharply away from the vertical between ground and first floors.

The building is founded on a circular helical concrete raft 8 ft. 6 in. (2.6 m) thick which supports both the central service core and the eight perimeter columns.

There are showrooms at ground level, office floors between the 1st and 20th floors, a restaurant on the 21st floor, kitchens and plant rooms on the 22nd floor, lift motor rooms and storage on the 23rd, and water tanks and condensers at roof level. The total floor area is 272,000 sq.ft. (2527m²). Oil storage and further water storage is located below ground, together with the electrical switchgear and transformer rooms.

Externally, the building is to be clad with mosaic facing and anodized aluminium windows are fitted.

The building is fully air conditioned by the use of individual *Versatemp* units fitted one per 8 ft. 6 in. (2.6 m) length of window, capable of individual and automatic adjustment for heating and cooling.

Fig. 1
Aerial view of the site
during excavations.
(Photo: Handford Photography)

Fig. 2 below
Typical floor plan.

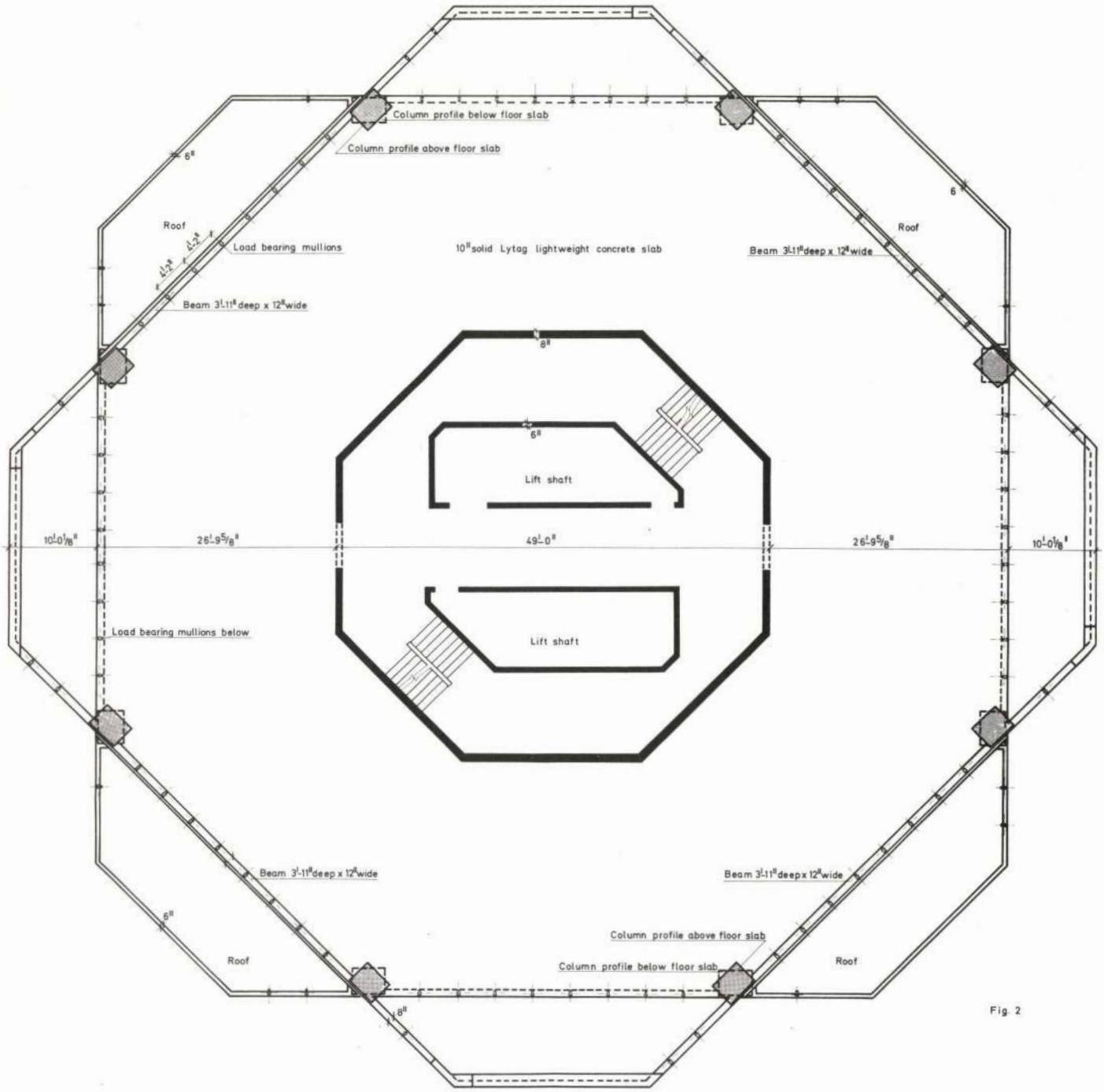


Fig. 2

Diaphragm wall

The excavation of a hole 36 ft. deep x 180 ft. diameter (11 m x 55 m) to accommodate the spiral car park was achieved by using a diaphragm wall. The outside radius of the wall is 89 ft. (27.1 m) in general, increasing locally to 91 ft. 6 in. (27.9 m) where vertical ducts are required to vent the car park. The wall is 1 ft. 8 in. (500 mm) thick and 60 ft. (18.3 m) deep, and was cast in 15 ft. (4.6 m) panel lengths. Casting was done by tremie pipe through bentonite drilling mud. It extends 30 ft. (9.1 m) below the base raft through the clay and acts as a cut-off wall against possible water-bearing seams, and also to guard against the possibility of ground heave due to the removal of overburden, prior to casting the base.

Individual wall panels have no capacity to transmit moment horizontally, only thrust, and have been designed as individual vertical beams to resist earth pressure.

It was decided that traditional propping during excavation could be eliminated and the diaphragm wall stiffened by casting three turns of a helical stiffening ring 5 ft. wide x 1 ft. 6 in. thick (1.5 m x 457 mm). The ring executes 2⅞ revolutions in spiralling down from level 201.00 to level 175.00 at a slope of 1 :48, and, at any given section, provides three horizontal supports to the wall at 9 ft. (2.74 m) centres vertically (Fig. 5).

The stiffening ring was cast in situ using the unexcavated ground as propping for plywood sheets laid on the ground (Fig. 6). Concrete was then placed on the plywood and into rebates in the diaphragm wall. These were formed by attaching polystyrene blocks 2 ft. x 3 in. (610 mm x 75 mm) to the reinforcement cages of the diaphragm wall before they were lowered into position through the bentonite.

To ensure monolithic action between the outstanding leg of the ring and the vertical panels, the rebate exposed on excavation was grit blasted after removal of the polystyrene inserts. U bars bent along the face of the panel were bent out to form a mechanical key.

The horizontal stiffening ring was itself supported by temporary raking ties of 1⅛ in. (28 mm) diameter mild steel every 15 ft. (4.6 m) approximately. The upper end of each raking tie was hooked through a hole and cast a bond length in a rebate left in the ring immediately above that which was to be cast and adjacent to the diaphragm wall. The holes and rebate were then filled with concrete. The lower end of the tie was set at the inner edge of the stiffening ring, and was cast in with the ring. This method of fixing ties was found to be the simplest and cheapest, and eliminated the need for costly turnbuckle adjusting screws. Alan Tricklebank, who was responsible for the structure up to ground floor level, spent many hours investigating alternative methods before the system used was finally evolved. A rebate was formed at the inner edge of the ring and reinforcement left projecting. This later formed the seating for the 10 in. (250 mm) thick car park slab. This junction acted as a day joint in the suspended slab after the remainder of the car park slab had been cast and the raking ties cut off. The contractor estimated that the elimination of traditional propping had reduced the excavation time, and his costs by £40,000.

Post construction comments

The tolerances quoted for the construction of the diaphragm wall were :

± 2 in. (50 mm) out of plumb.

These were not achieved. At 30 ft. (9.1 m) below ground level (maximum exposed depth of wall) the variation from the mean radius was from − 12 in. (305 mm) to + 20 in. (510 mm), the latter figure indicating that the particular panel had not made any contact whatsoever with the adjacent panels on either side, thus destroying completely the ability to withstand compression at that point.

Before excavation proceeded further this was made good by casting a new wall onto the face of the offending panel.

Excavation of the slit trench for the diaphragm wall was through gravel. In some cases pockets of gravel fell into the bentonite, leaving a void to be filled, first by bentonite and then concrete, thus causing large concrete lumps projecting approximately 6 ft. (1.8 m) out from the inner face which had later to be cut off.

At one point during excavation the excavating bucket cut through an old sewer, and all the bentonite disappeared down it, rather in the manner that bath water disappears when the plug is removed. Attempts were made to plug the sewer by using lean mix concrete and this was eventually successful. Clay, if available, is the normal means but in this instance the excavation was through gravel and sand.

Stiffening ring

The following assumptions were made for the analysis of the ring :

1 The stiffening ring would act as a coiled spring within the drum.

Fig. 3
Architectural model of the development.
(Photo : Fox Photos Ltd.)
Reproduced by permission of R. Seifert and Partners.

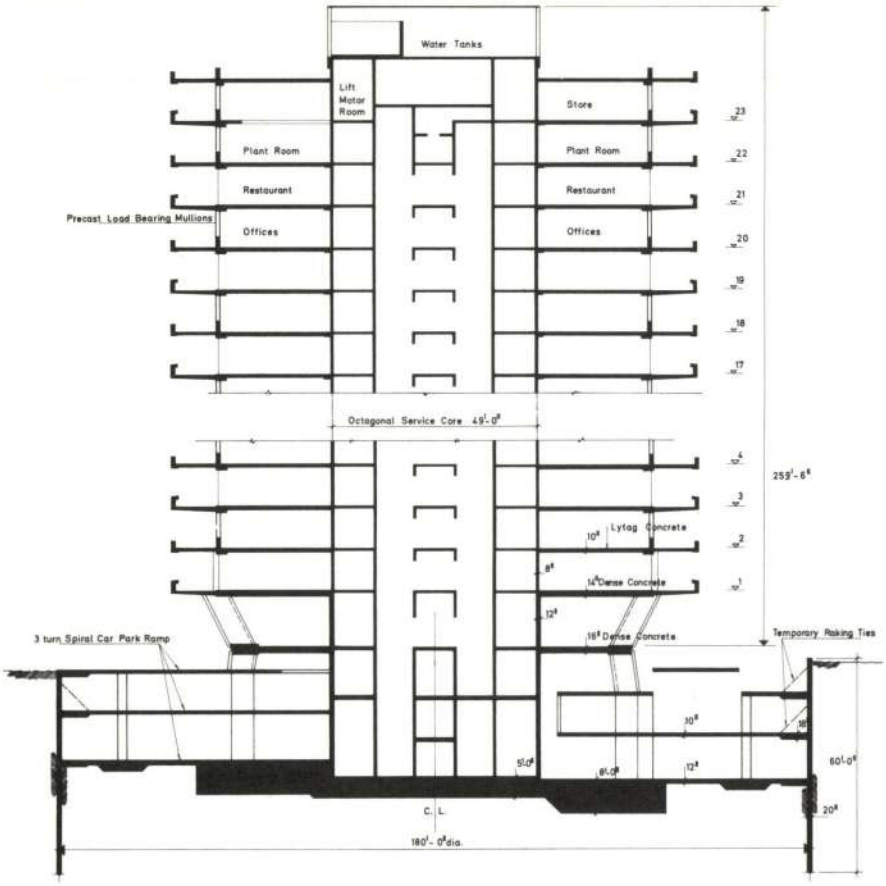


Fig. 4 left
Vertical section through the building.

Fig. 5 below
Construction of helical stiffening ring.

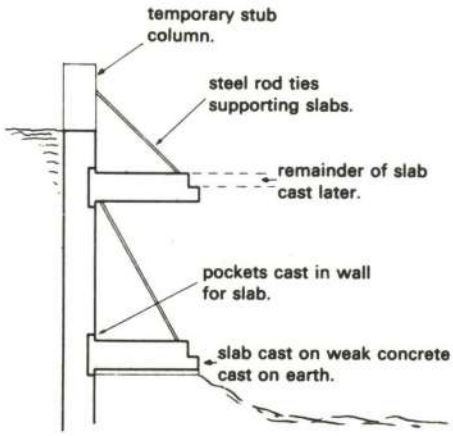


Fig. 6
Excavation of car park showing stiffening ring (cast on ground) supported by raking ties.
Excavation was entirely by mechanical equipment with complete elimination of any temporary propping.
(Photo: E. J. Studios). Reproduced by courtesy of Taylor Woodrow Construction Ltd., who are the main contractors.

2 Uniform load would be exerted radially onto the spring.

3 The spring would be infinite in length.

4 The 5 ft. (1.5 m) stiffening ring would act compositely with the vertical diaphragm wall panels to form a T beam lying on its side with a 9 ft. (2.7 m) long x 1 ft. 8 in. (510 mm) thick vertical leg and a 5 ft. (1.5 m) long x 1 ft. 6 in. (460 mm) thick horizontal leg.

If the drum were completely circular the forces in it arising from earth pressure would be solely compressive. At each of five points around the ring, the ring is pushed out 2 ft. 6 in. (760 mm) radially over a 15 ft. (4.6 m) panel length, returning to the mean radius over the length of the adjacent panels. At this point the ring is pierced by irregular holes each approximately 3 ft. 6 in. x 2 ft. (1.1 m x 610 mm). This disturbance of shape from the circular induced bending and shear, as well as compression, into the ring.

The conditions of stability investigated were:

1 Stability of the drum as a whole

$$\text{Euler buckling load} = \frac{\pi^2 EI}{r^2}$$

The factor of safety was 8.

2 'Snap through' buckling of the drum (1)

$$\text{Theoretical buckling load} = \frac{80EI}{a^3}$$

The factor of safety was 11.2.

3 Local buckling (vertically) of the stiffening ring between raking ties.

$$\text{Euler load} = \frac{\pi^2 EI}{L^2}$$

Factor of safety was 8.3.

Foundation raft and car park

The raft, which supports both the central core and eight perimeter columns, is 108 ft. (32.9 m) in diameter and 8 ft. 6 in. (2.6 m) thick. It is formed as a spiral rising 9 ft. (2.7 m) in one complete turn. This forms the first turn of the car park.

Excavation

The raft sits on sand which, when drained, is hard. During excavation, as the spiral proceeded downwards, a sump was located at the lowest point and kept pumped out. The lowest point of the underside of the raft is some 36 ft. (11 m) below road level, and 24 ft. (7.3 m) below the water table. Excavation was carried out by drag line and front loading shovel and took two months, from February to March 1968, a fairly dry period, and little or no serious softening of the ground was experienced due to rain. Drainage of ground water within the drum down to the sump was noticeable, and, as excavation proceeded, vertical sand faces, left at higher levels, dried out and became very hard.

At the lowest levels the sand surface became damp within fifteen minutes of excavation,

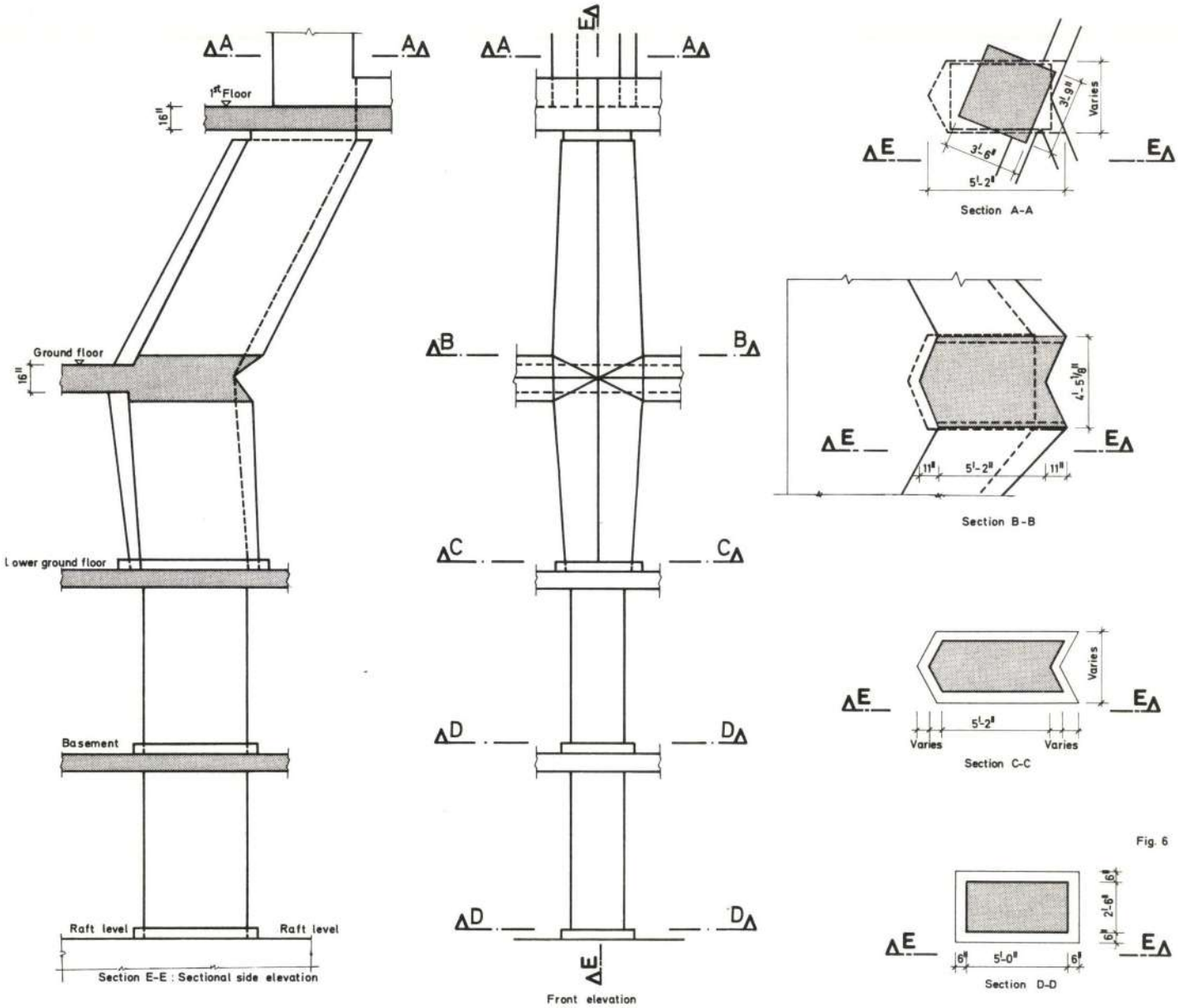
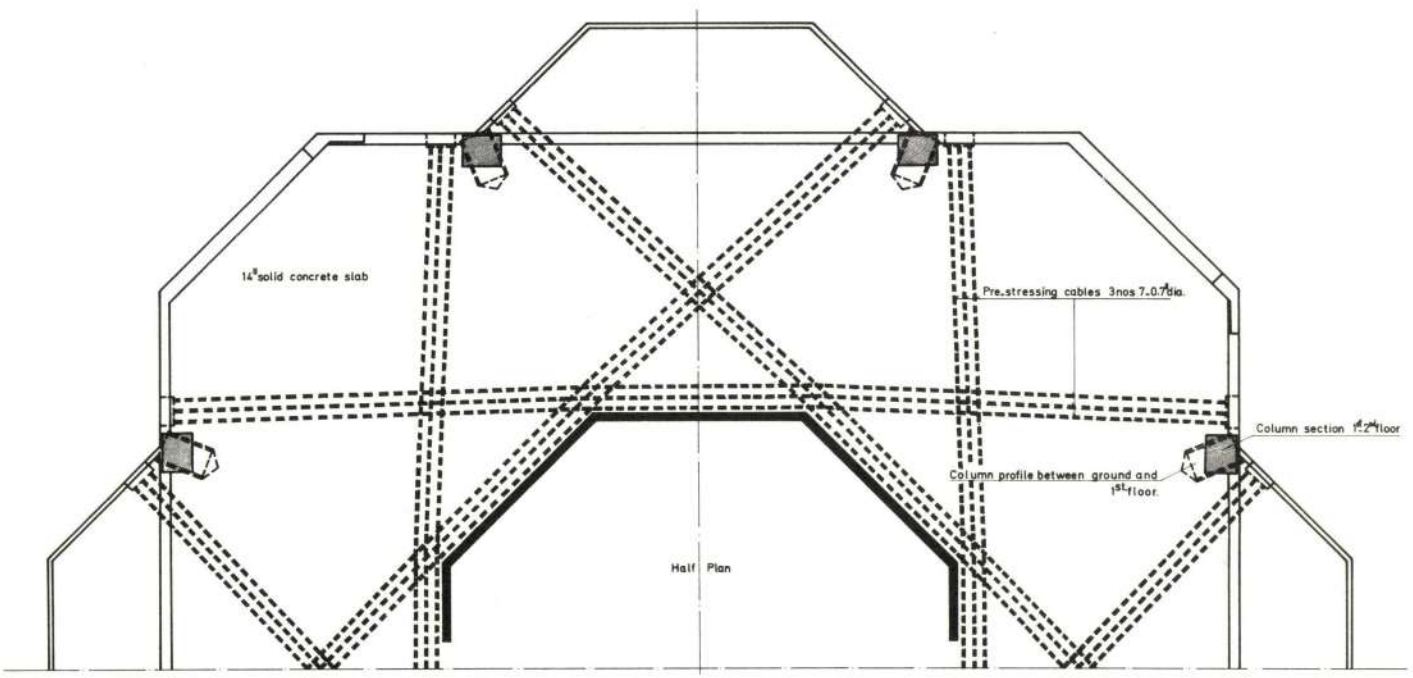


Fig. 6

Fig. 7
Elevation and sections of the sloping columns between the raft and first floor.

Fig. 8
First floor layout of prestressing cables.



and patting it with the flat of the hand would bring water to the surface. Locally, as soon as the excavation was completed to final level, it was covered with 2 in. (50 mm) blinding. The diaphragm wall was very effective in cutting off ground water from outside the drum and little water seeped through the joints. A water main in the ground, immediately outside the wall, fractured and there was no indication of this burst on the inside face of the wall. There was no evidence within the drum, at any time during excavation, of either piping or ground heave.

The weather gods were indeed kind during this period and far more rapid progress was made and fewer problems developed than we had expected.

Design

The diameter of the raft was varied during design in order to equalize, as much as possible, the radial and circumferential bending moments.

For purposes of analysis, the raft was considered as a circular plate and the screw effect ignored. A sector of the circle was analysed on the computer using the finite element approach. John Storrs, who was responsible for the raft design, found, after some trial, that it was possible and more convenient to do this using a radial and circumferential grid instead of the more usual rectangular grid.

Main columns

Eight columns disposed at the points of the compass at the perimeter of the building share the load of the building almost equally with the central core. The central core is an octagon 45 ft. (13.7 m) across the flats which accepts all the wind loading.

From raft level upwards, through two floors, the columns are rectangular, vertical, and continuous in section. Between lower ground floor, ground floor and first floor they have a constantly changing chevron shaped section and no longer remain vertical, sloping in towards the centre 9 in. (230 mm) between lower ground and ground floor, and sloping outwards 6 ft. 9 in. (2.1 m) (23° away from the vertical) between ground and first floor (Fig. 7). Above this level they are vertical.

At first floor level the sloping columns are tied into the building by a system of prestressing cables laid at mid depth of the 1ft. 2 in. (360 mm) thick dense concrete floor slab. Six cables restrain each column and exert a total radial tie force of approximately 1800 kips (Fig. 8).

If one has a building which abuts major roads on all sides and the grouting operation for the cable ducts is 13 ft. (4 m) above road level, and the grout pressure is 100 psi (0.69N/mm²), one should remember that a horizontal grout hole in the end plate can produce a most impressive grout fountain into the middle of the main road. We forgot until almost the very last moment. It might have been fun, even if it had proved expensive on re-sprays for passing cars. A grout deflector plate was cheaper, even if it was more mundane.

The 16 in. (410 mm) thick ground floor slab acts as a thrust ring to prevent the feet of the sloping columns from moving inwards.

The columns were originally envisaged as being reinforced concrete. Mid-way through the design stage the shape of the columns changed and they were made to slope both away from, and then back towards, the vertical. This increase in slope, and the change in slope, presented problems of reinforcement detailing. Also each sloping column was different in length from every other sloping column.

We were already using 8000 psi (55.2N/mm²) concrete in order to achieve the column size required. It had been intended to use 1½ in. (38 mm) diameter high tensile reinforcing bars and 36 bars would have been needed. This was too many, particularly at the splices. A 2¼ in. (57 mm) diameter high tensile bar was chosen and this gave a reasonable layout of steel.

Fig. 9
Cranked column reinforcement formed by two 12 in. x 12 in. universal columns and eight 2¼ in. diameter *Hi-bond* bars. (Photo: Peter Ryalls)

The design change in column profile and slope increased the number of bends in the reinforcing bars, entailed bends in two planes and made it practically impossible to locate the bars in their correct position at first floor level, at which point the column returned to the vertical. Each bar end at first floor level was required to be placed to an accuracy of ± ¼ in. (6 mm) and the contractor was required to use templates to achieve this.

The bar manufacturer was unable to produce a bar some 28 ft. (8.5 m) long bent in two planes with the required degree of accuracy.

Thus the problem to be solved was that of providing reinforcement which could be accurately placed in all three planes. It was decided that reinforcement to the sloping parts of the column should be two 12 in. x 12 in. (305 mm x 305 mm) universal columns, cranked and welded at the change of slope and strapped together to form the main reinforcement (Fig. 9). The universal columns were butt welded to machined base plates top and bottom (Fig. 10). Corner bar reinforcement was provided by four 2¼ in. (57 mm) bars also butt welded to the base plates. Shear connector links were welded to the universal columns and conventional links were attached to the 2¼ in. (57 mm) bars.

Because of the welding and the thickness of the sections and the difficulty of normalizing a fabricated element 28 ft. x 4 ft. (8.5 m x 1.2 m), the universal columns were specified as ND (notch ductile) 2b steel and the 2¼ in. (57 mm) bars were McCalls weldable *Unisteel* '60'. This part of the work was put out to tender to steel work fabricators.

The part of the fabrication which most disturbed some tenderers was the bending of the links. This was outside their experience and worried them far more than either the accurate setting out or the machining or welding.

It was considered impossible to test the welds on the steelwork—(X-rays of welds are only satisfactory on butt welds) and a firm of

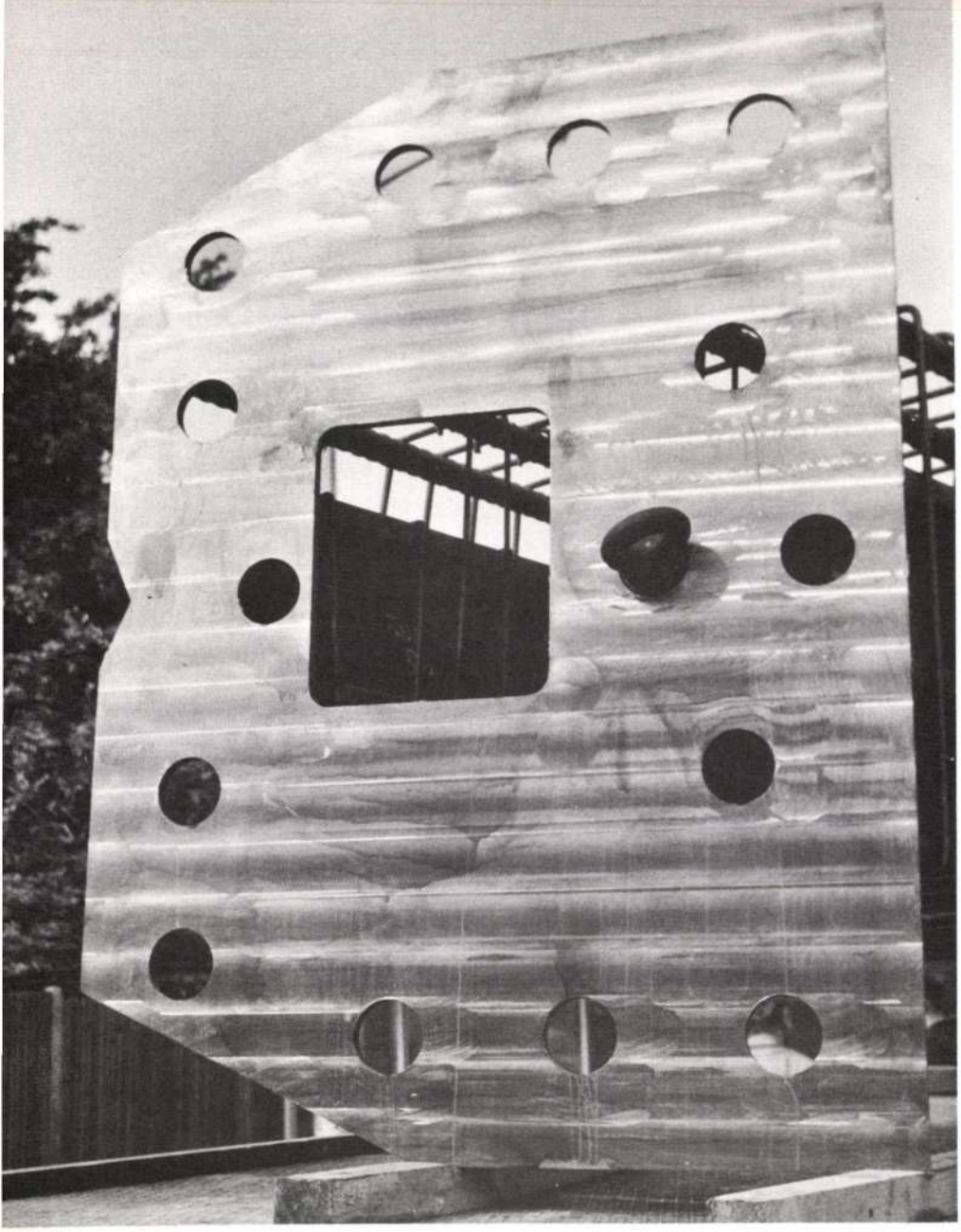
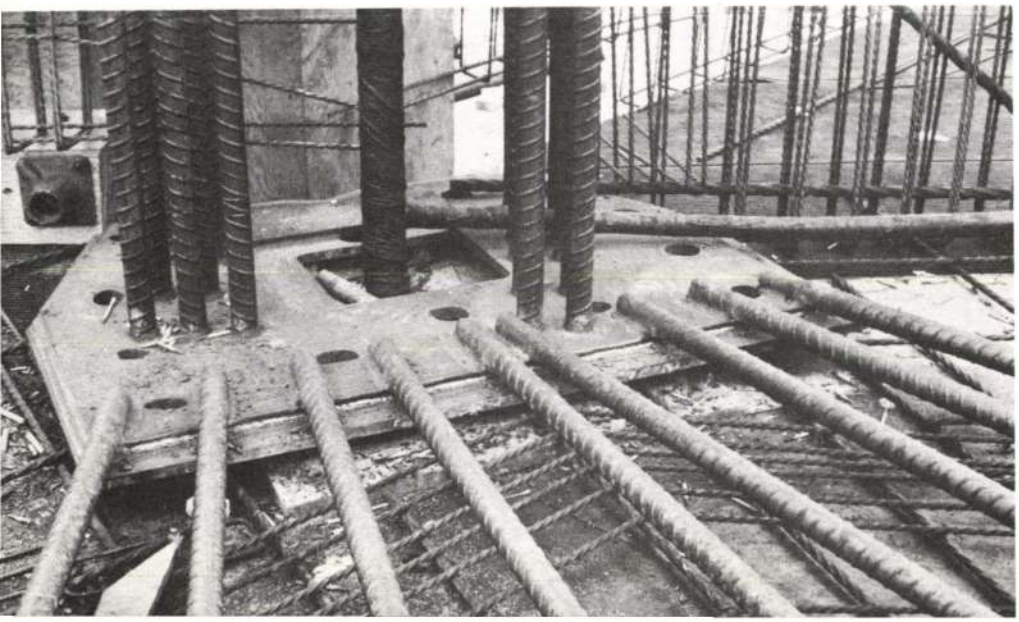


Fig. 10
Upper plate of fabricated column. (To this plate a loose plate containing 22 starter bars is welded at first floor.)
(Photo: Peter Ryalls)

Fig. 11
Column at 1st floor level showing loose plate with 22 2¼ in. diameter vertical bond bars and nine horizontal 2¼ in. diameter *Hi-bond* bars welded to it, and loose plate welded to upper plate of the fabricated column. The 6 in. diameter rain water pipe is also shown passing vertically through a rectangular hole in centre of plates.
(Photo: Peter Ryalls)



welding inspectors, Corrosion, Welding & Engineering Ltd., was employed to test the welders initially and to supervise the actual welding. They also supervised the site welding.

Between foundation level and lower ground floor each column is vertical, rectangular in shape and reinforced conventionally with 2¼ in. (57 mm) bars. To provide a junction between the reinforced concrete column and the steel column, a loose base plate was used. This plate was accurately levelled and bedded at lower ground floor level. 2¼ in. (57 mm) bars projected through holes in the plate. They were cut off flush with the top of the plate and the annular space between the bar and the edge of the hole filled with a dry pack mortar. At this level, tension does not develop in the column and a tension connection of the loose plate to the bars is not required. The steel column was then lowered onto this plate and welded to it.

At first floor level a loose plate was welded to the upper end of the steel column. To this loose plate, 22 2¼ in. (57 mm) bars were welded to act as starter bars for the column above the first floor (Figs. 11 and 12).

Columns above 1st floor

Above 1st floor each column is vertical and rectangular in plan (Fig 13). At each floor the plan rotates through 45°, and the section reduces progressively up the building. On each successive floor a main edge beam frames into the column, on the opposite side of and 45° inclined to both that on the floor above and that on the floor below. In four of the eight columns a 6 in. (150 mm) rainwater pipe passes down the middle and collects water from balconies on alternate floors.

The plan area of column overlap at slab level is termed the common core. This is the area through which the vertical load of the column must be transmitted. This area, as a percentage of the total area of the column, varies from floor to floor. Due to the manner in which the moments are applied from the main edge beams, slab level is a point of contraflexure, and the common core is designed for vertical load only without bending moments. Because only part of the plan area of the column is loaded at each floor, i.e. the common core, the column in the region immediately above and below the floor slab, behaves as a prestressed end block, and was analysed accordingly ([2 3]). The column tends to split down its length and develops high horizontal splitting forces which must be resisted by binders. The loaded area is not rectangular, and an equivalent rectangle was used for analysis in order to determine these forces.

Early investigations indicated that at second floor level 18 sq. in. (11,600mm^2) of binding steel would be required to resist splitting.

In order to check the validity of the assumptions, Professor Brock at Loughborough University, in co-operation with Sakda Bunyaraksh of our office and later Stein Ingolfsrud, undertook a number of model tests using 1/12 scale models of the column and its beams. Each model was made of plaster of Paris and was reinforced in the same manner as the actual column using longitudinal reinforcement and binders to scale. To achieve proper bond the longitudinal reinforcement was threaded over its full length.

Five tests were run using the same longitudinal steel but varying the number and spacing of the binders.

The tests, whilst not exhaustive, indicated that:

1 Doubts about the original analysis were unjustified as the column failed by tensile splitting rather than crushing.

2 The ultimate tensile splitting failure occurred at a load considerably less than the calculated ultimate crushing load.

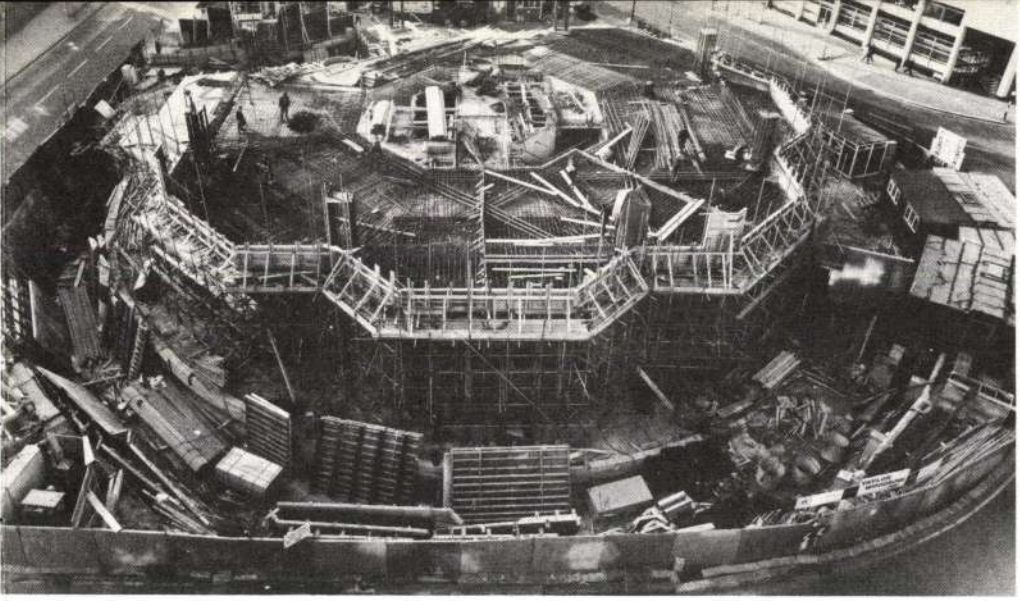


Fig. 12
1st floor level showing column reinforcement erected between 1st and 2nd floors and templates attached to the reinforcement. Some stressing cables are shown in position.
(Photo: E. J. Studios). Reproduced by courtesy of Taylor Woodrow Construction Ltd.

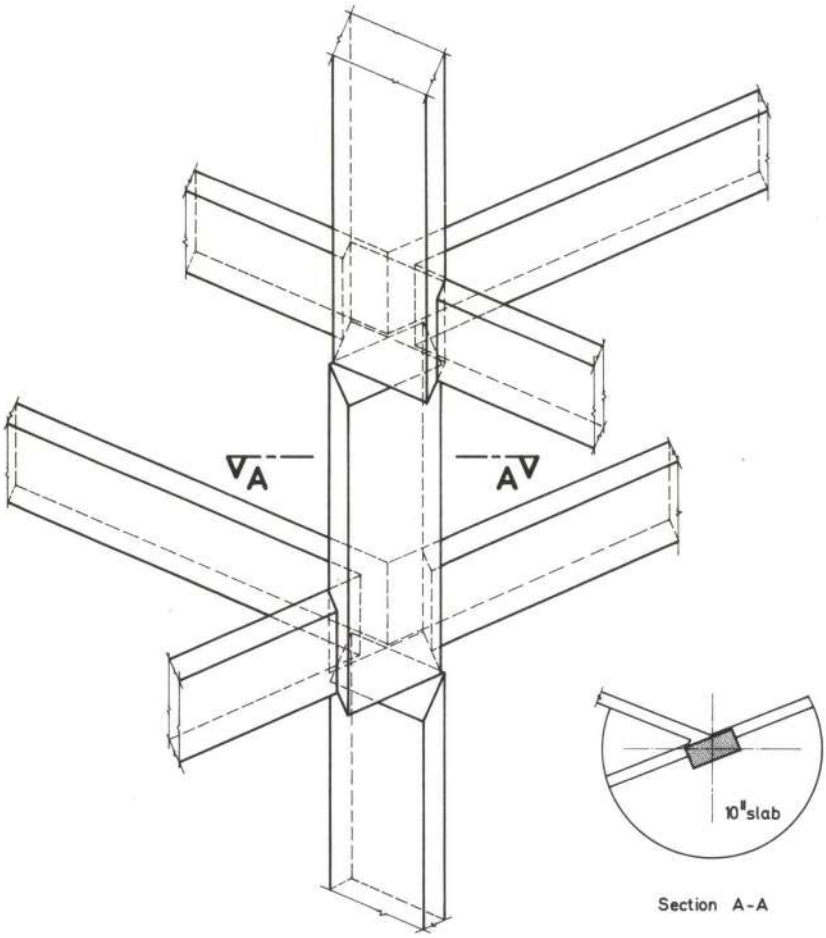
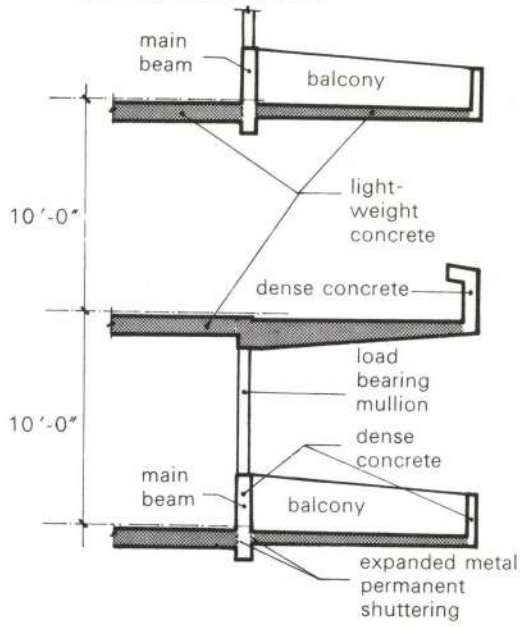


Fig. 13 left
Diagram showing the rotation of columns floor to floor.

Fig. 14 below
Section through mainbeams showing the dense concrete beam projecting through the lightweight concrete slab.



3 There was an effective limit to the amount of binding steel, and above this limit no increase of ultimate load was achieved.

In practice, the largest amount of binding steel used at one level was 19 sq. in. (12,260 mm²) (⅝ in. (16 mm) diameter mild steel in groups of four at 4 in. (100 mm) centre to centre to resist a tensile splitting load of 772 kip induced by a vertical load of 4000 kip.

The binders were an odd shape (and changed shape from floor to floor) to accommodate vertical steel passing through the common core and ending up in the middle of the column above, and the reinforcement manufacturer demanded that these links be drawn full size at all levels, and charged an extra for the bending.

If on any future job the use of large diameter bars is contemplated, i.e. 2 in. (50 mm) or 2¼ in. (57 mm) diameter high tensile bars, one may find, as we did that:

There are no regulations governing their use in CP114. Referring to the American code of practice it will be found that:

for bars of 1⅜ in. (35 mm) diameter and above mechanical butt splices are mandatory. Mechanical butt splices require the ends of bars to be sawn square (not cropped) to an accuracy of ± 1°. This may be quoted as an extra of £5 per ton for the sawing only.

2¼ in. (57 mm) bars are rolled in 50 ton batches at rather infrequent intervals. If less than 50 tons are wanted it will probably be necessary to wait until someone wants the remainder. If 50 tons are wanted then this order has to be fitted into the rolling programme at the works which may, or may not, be fitted in quickly.

It may then be found, because the contractor has no experience of the *G-Loc* mechanical butt splice, that his steel fixers are demanding high rates for fixing them, although he has to some extent offset this by putting a very high rate in the bill. On site the bars are in fact fixed easily, rapidly and cheaply. One may, like us, then breathe sighs of relief, and decide that large diameter bars in this country have large disadvantages in the supply situation, but can be absolutely invaluable where there is otherwise a need to splice 36 1½ in. (38 mm) diameter bars.

Construction problems with the columns

At the beginning of the job, because of the very limited space on site and the number of different mixes required, the contractor decided to use ready mixed concrete for the 8000 p.s.i. (56N/mm²) mix only, the nearest plant being ½ mile away. The ready mix plant only had one bin available for the coarse aggregate and thus was permitted (against our better judgement) to use a ¾ in.–⅜ in. (19 mm–10 mm) combined granite aggregate. Satisfactory preliminary strengths were produced.

Although the plant was only ½ mile from site, it took on occasion ½ hour for the concrete to

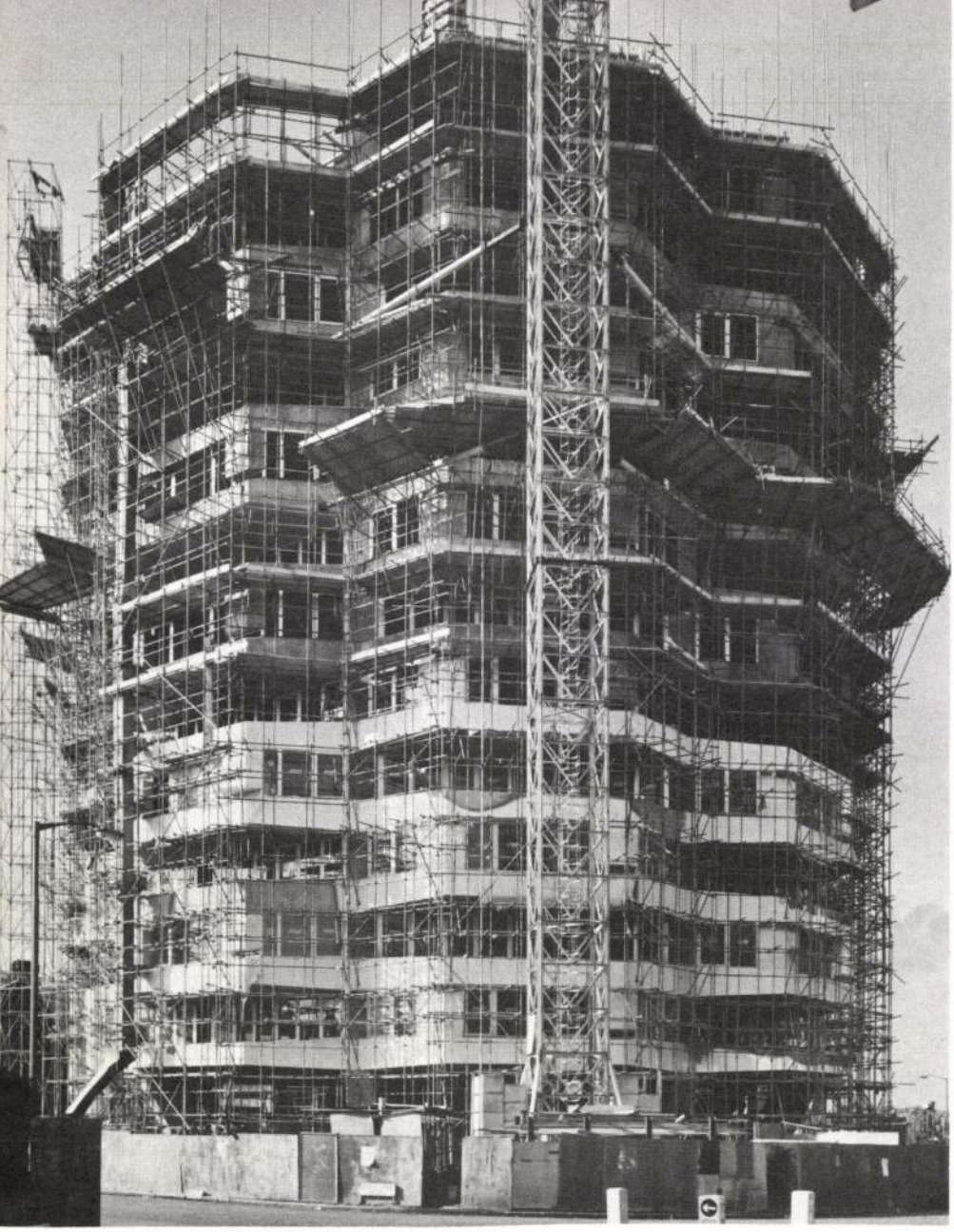


Fig. 15
View of building at end of September 1969, with construction at 13th floor.
(Photo: E. J. Studios). Reproduced by courtesy of Taylor Woodrow Construction Ltd.

arrive; and when it did it would not leave the truck or if it did, would not then leave the skip. Site mixing was introduced and casting proceeded up to 1st floor. Strengths were never high enough to eliminate concern, and every single cube result was examined with some measure of anxiety.

Attempts were made to increase the average strength and every aspect of the concrete was examined. The cement was variable, but the seven day results were around the 4000 mark (28N/mm²), and the suppliers could not guarantee a minimum strength cement, or even a consistent one.

The contractor agreed to use single size coarse aggregates in place of the combined stockpile as a means to eliminate one element of variation. In addition, an alternative supply of granite was made known to us—Penlea granite. This has been used for the room at the Tower of London containing the crown jewels where 10,000 psi (70N/mm²) at 28 days was required. This was substituted for the Manuelle granite previously used, and stored in separate bins. The seven-day results, using the new aggregate, improved immediately, which seemed to justify the change, but so did the last set of 28-day results using the old aggregate—which threw suspicion back again to the cement. However the results using the Penlea granite have been consistently good, the average 28-day cube strength is 9500 psi (67N/mm²) with an aggregate/cement ratio of 3.4, a water/cement ratio of 0.34 and a slump of 2 in. (50 mm) achieved by the use of *Sealoplaz* plasticizer, and the standard deviation is 800 and this concrete has now been relegated to the ranks of 'ordinary'.

If only we could have started at the top of the building and worked our way down!

Typical office floors

There are 20 typical floors (Fig. 2) and each is a 10 in. (250 mm) solid flat slab disc of lightweight *Lytag* concrete supported at the centre by the service core walls, and at the outer edge by four perimeter beams formed of dense concrete and four rows of load bearing mullions at 4 ft. 2 in. (1.27 m) centre to centre.

Various floor systems were investigated. The plan shape precluded the use of either hollow tile floors or ribbed floors because the ribs would be radial and vary in width.

This left two possible systems:

1 a 10 in. (250 mm) solid lightweight slab, or
2 a 10 in. coffered slab.

As it was uncertain which would be cheaper, both were put in the bill of quantities and tenderers given the opportunity to price both.

Of the seven tenders received, five priced the solid lightweight slab cheaper than the coffer, as did the successful tenderer.

Although material costs of the lightweight concrete are greater than dense concrete, (aggregate costs being 15% greater than gravel in the London area and the lightweight concrete being priced at 9/– per cu. yd. more than the dense), the great saving in shuttering costs and incidentally in time (approximately £1 per sq.yd. cheaper than the coffer formwork) produced an overall cost differential of 6/– per sq.yd. in favour of the lightweight slab.

The main beams (Fig. 14) project below the slabs and this created a construction problem for the contractor (Fig. 13). To separate the lightweight slabs from the dense concrete beams, expanded metal mesh permanent shuttering has been used. The lightweight slabs are cast before the beam and the *Lytag* is cast against the expanded metal which serves both as shutter and shear key. The reinforcement from the slab pokes through the expanded metal, through the beam, and through the expanded metal on the other side of the beam.

One disadvantage of using lightweight concrete slabs is the difficulty of applying, to the soffit, a plaster finish which will remain permanently in place—it has a very nasty tendency to drop off. The porosity of the slab tends to suck the moisture out of the plaster before it has gone off. To date we do not know of anyone willing to guarantee a plaster finish to a lightweight concrete slab.

To be informed, on first acquaintance with lightweight concrete, that the best way to control the water is with a good mixer driver is a little disconcerting. Practice, however, proves the point. The aggregate, being porous, can arrive on site with widely varying moisture contents (5–15%) and the mixer driver is indeed the best man to adjust this. It is always possible to tell if the mix is too wet—the aggregate pellets simply float to the top and the surface looks as though it has been visited by a large number of rabbits.

Lightweight concrete data

Specified 28-day cube strength:	4200 psi (29 N/mm²)
Aggregate/cement ratio:	4.5
Water/cement ratio:	0.53
Mix proportions	
Lytag dry volumes	
Fines:	10.6 ft.³ (0.3 m³)
½ in. medium:	21.4 ft.³ (0.61 m³)
Cement:	655 lb. (297 kg)
Aggregate densities:	45–60 lb/ft³ (720–960 kg/m³)
Concrete: saturated density:	115 lb/ft³ (184 kg/m³)

Construction at the time of going to print (Fig. 15) is at 13th floor and we are working on the design of the roof tanks, plant rooms and restaurant area (which may yet be omitted).

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The design and construction of Sheffield University Concourse

Keith Ranawake, Bob Gordon and Ken Tune

Background to the project

The last decade has seen the rapid growth of universities in this country and Sheffield University has followed this pattern. The geography of the space available dictated that this expansion should occur along, and to either side of, a major road artery into the city. The equally rapid growth of vehicular traffic also resulted in the replacement of the old single carriageway road by a dual carriageway. Thus the university found itself bisected by a wide heavily trafficked road carried on an embankment, over which up to 10,000 pedestrian crossings per day were made during term time. This situation was totally unsatisfactory from the points of view of both the university and the City Engineer, who had to face the disruption of vehicular traffic caused by the pedestrians. The university therefore prepared a brief in which a possible solution to the problem was outlined. The brief required that there should be safe and unrestricted movement from one part of the university to the other and that the crossing to be provided should have a definite amenity value. We were approached in the summer of 1965 and appointed as consultants in January 1966.

The overall design

The key to the functional solution of the problem lay in the recognition that a crossing could be made on a plane passing under Western Bank which connected the lowest level of the buildings to the north with the open area to the south. This plane contains the main lines of pedestrian flow, as long as adequate access points are provided to it. The virtually complete isolation of the pedestrian and vehicular traffic from each other is thus possible. Some re-alignment of the dual carriageway would be necessary but this, fortunately, improved upon the existing alignment. Estimates of cost confirmed the economic feasibility of the scheme.

The aesthetic requirement that had to be satisfied before any functional solution could be accepted was that the connecting element should have a feeling of lightness and space and that it should integrate and give a sense of unity to the two halves of the university.

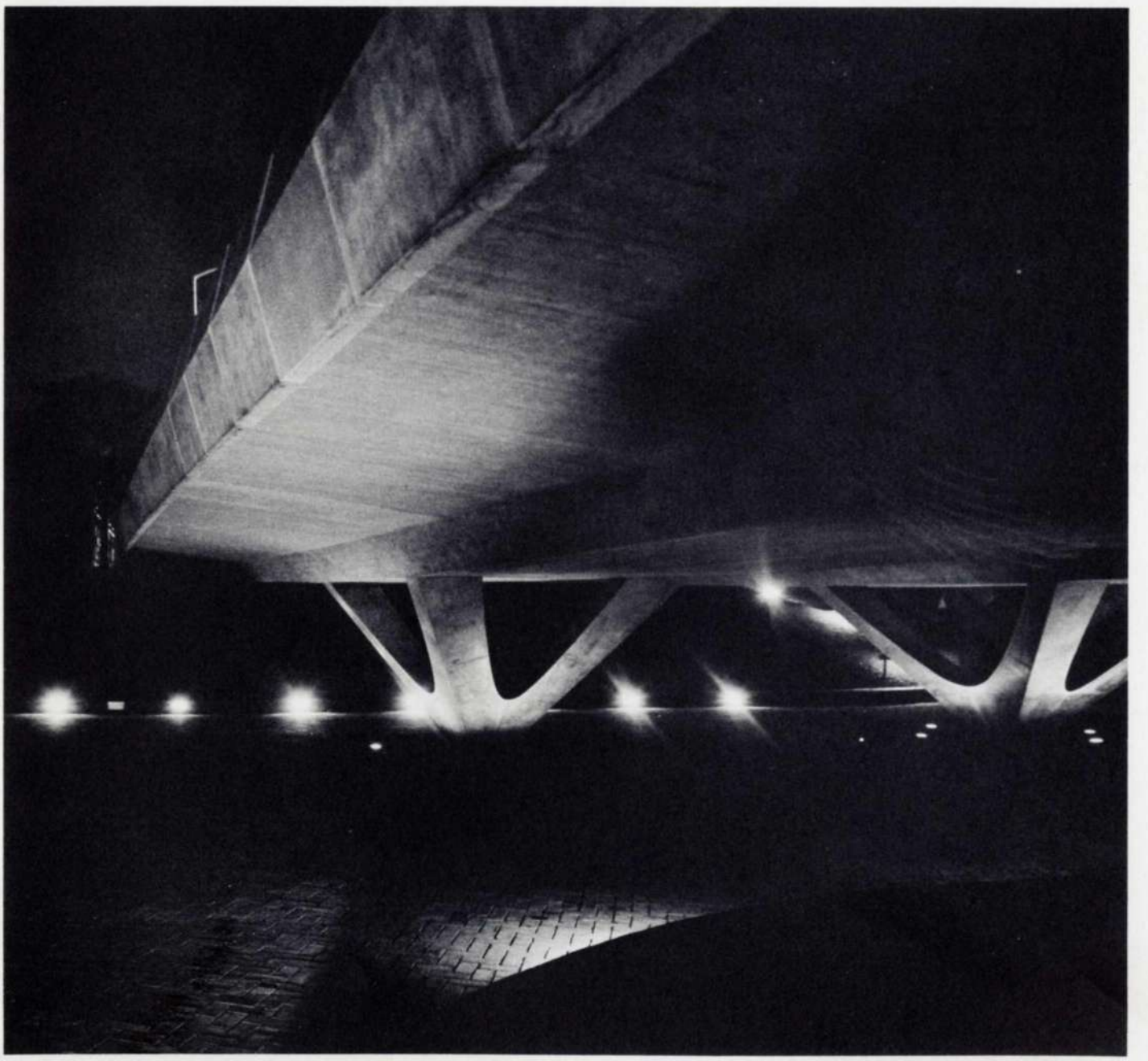
The aim has therefore been to create a design which has none of the characteristics of a tunnel and where landscaping to both sides would provide a space more in the nature of a garden, with an informal and freely flowing outline to the paved area.

Design and analysis of the bridge

Design

In this design a conscious attempt has been made to avoid the directional effect which is often felt under wide bridges. It was therefore essential to avoid any vertical abutment walls

Fig. 1
'Indirect lighting is used at night to guide pedestrians.'
(Photo: Henk Snoek)



or edge supports which would create a visual tunnel. In addition, there existed the functional requirement that the bridge had to be constructed in two separate halves as one half of the dual carriageway had to be kept open to traffic at all times.

Various ideas were developed and discussed. The design which eventually emerged was included in our report to the university in November 1966 and was accepted. Yuzo Mikami was the architect.

Description of the bridge structure

The bridge is, in fact, two separate bridges with a lightwell between. Each deck is a continuous, solid reinforced concrete slab on a 5% gradient. The central spine of the slab tapers from 4 ft. (1.2 m) deep over the length of the central support to 2 ft. 9 in. (840 mm) at the abutments. The central spine is straight and the horizontal curvature is taken out in the edge cantilevers. The central support consists of four raking legs, monolithic with the deck and coming down to join immediately above a single 1000 ton bearing which approximates in action to a spherical joint. The outside plane faces of the central support are the faces of an inverted truncated pyramid with a rectangular base. The inside curved faces are formed by the intersection of two orthogonal hyperbolic surfaces. Sections through each leg in any plane perpendicular to the central axis of the support are identical rectangles whose proportions vary with the position of the plane. Sections perpendicular to the leg axis are irregular quadrilaterals. The central axis of symmetry is normal to the soffit of the deck over the support.

At the abutments, each end of the deck is supported on two 175 ton guided bearings which permit rotation about longitudinal and transverse axes and translation in the longitudinal direction only. The system of forces acting on each bridge through the bearings is shown in diagrammatic form in Fig. 4.

The foundation to the central support is a reinforced concrete buttressed pier on a spread footing, bearing on shale at 4 tons/sq. ft. (429kN/m^2). The abutment capping beams, also in reinforced concrete, sit on mass concrete piers also extending down to the shale.

Analysis

We decided to attempt to analyse the structure in parts and then build up a picture of the whole. We assumed that, as the central support assembly was very stiff, it would tend to act as a rigid body, rotating about the spherical joint. The question of lateral stability was of particular importance and we were concerned to know whether the predominant action would be the horizontal bending of the deck between abutments, or whether torsional bending was more significant. Under the action of out-of-balance transverse moments, due to eccentric loading of the carriageway, the length of deck over the central support would not experience torsional deformations if our assumption were correct. It would rotate as part of the central support assembly. This rotation would cause horizontal deflections and torsional rotations of the adjacent spans.

To test our assumption, we considered the loadcase where a 180 ton HB vehicle is positioned in the offside lane immediately over the central support. Energy equations were then used to calculate the deflection and rotation of the deck and hence the reactions at the bearings. We found that the lateral stability was mainly provided by the horizontal bending of the deck with accompanying transverse reactions at the abutments and the eccentric central support bearing. Torsional moments were relatively small. The horizontal forces calculated in this way were then applied to the deck by itself, spanning between abutments, and then to the central support alone, assuming encastré ends to the four legs. The deformation of the central support apex was approximately 10% of the deformation of the deck by itself. The moments in the four legs were also quite small, axial forces being the main actions. The initial assumption was therefore reasonably correct.

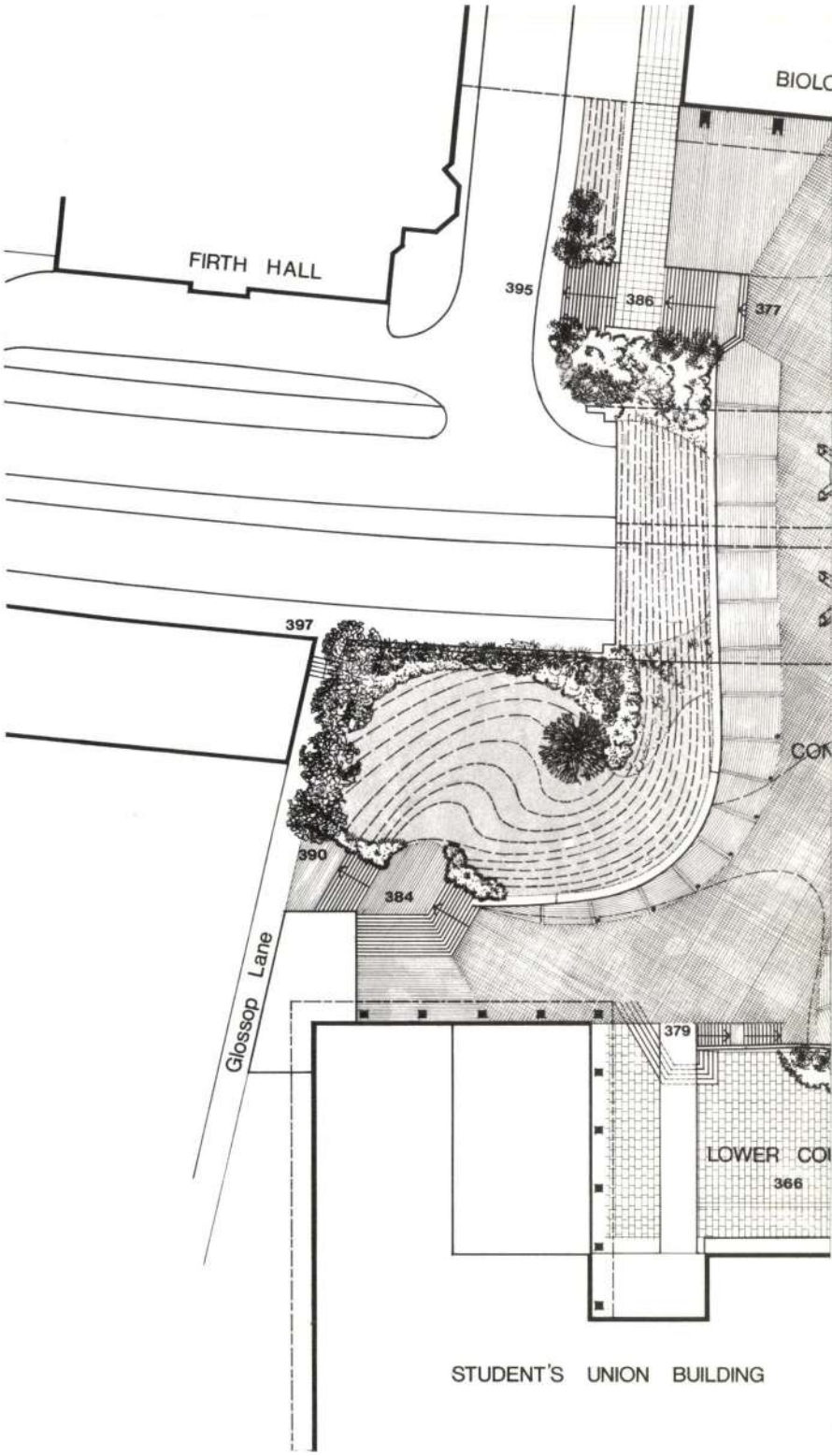


Fig. 2
Sheffield University Concourse.

The analysis then proceeded in four stages:

1 Analysis of the deck as a plate on point supports at the abutments and at the intersections with the four central legs. Grid analogy used. (Computer program OA 101.)

2 Analysis of deck and central support as a two dimensional frame applying vertical and horizontal loading. (Computer program OA 100.)

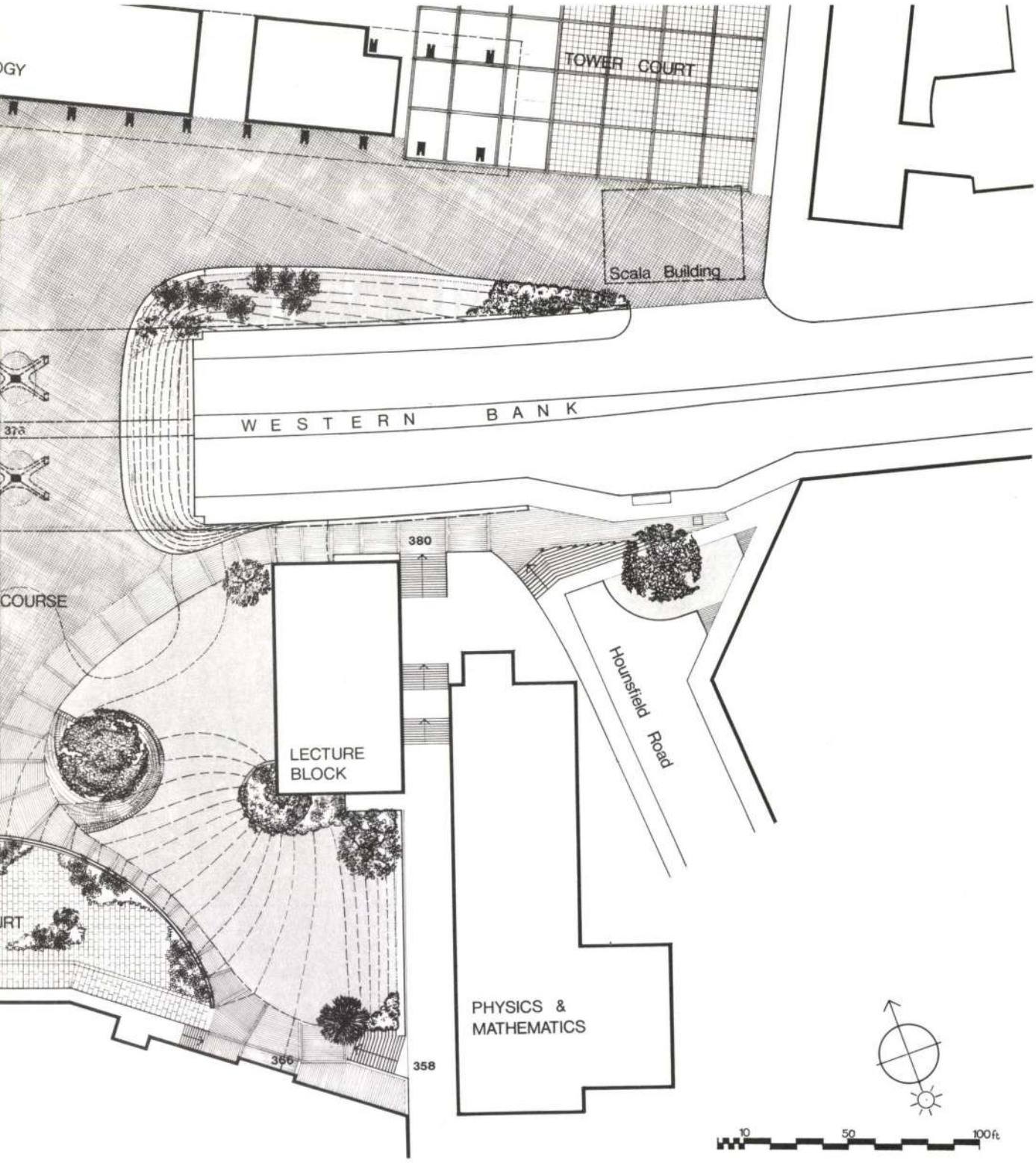
3 Using the reactions obtained to calculate the horizontal transverse reactions at all the bearing points for different loadcases.

4 Analysis of the central support by itself, assuming encastré ends to the four legs under the reactions derived previously. (Computer program OA 102.)

Typical longitudinal and transverse bending moments are shown in Fig. 5. The only significant deflections were those due to longitudinal bending. Maximum calculated vertical deflection is ⅝ in. (16 mm) approximately.

Landscaping

The design of the landscaping turned out to be the most difficult problem of all. This, despite



the fact that the basic solution proposed was never really in dispute.

The design, as built, is the third of three schemes and is the work of David Faggetter. The design should speak for itself and we shall limit ourselves to a brief factual description. The reader should note that no tree and shrub planting had been carried out when the photographs were taken.

The shape and extent of the main paved areas were determined by the direction of the pedestrian flows. Occasional vehicular traffic is permitted. Overburnt engineering bricks in herringbone and stretcher bond have been used. These provide a good non-slip surface, are reasonably priced and have a subtle range of colours. Low brick walls border the paved areas at the bottoms of the banks and provide informal seating. Drainage is to the boundaries of the paved areas. The surrounding banks are grassed and extensive use of tree and shrub planting has been made. Indirect lighting is used at night to guide pedestrians rather than to provide a high overall level of illumination.

Changes of level at access, and other points, have required the construction of various retaining walls, ramps and steps, all of which are faced in brick.

Construction

The project was let to tender in March 1968. Shephard, Hill & Co. Ltd. were appointed as general contractors in June 1968. Work started on site in July 1968 and was completed, except for the planting, in September 1969.

The contract commenced with the diversion of services and bulk excavation. The high concentration of services on Western Bank which had to be diverted under the concourse, coupled with the limited access to the site, created certain problems for the contractor initially. The weather in the autumn was also

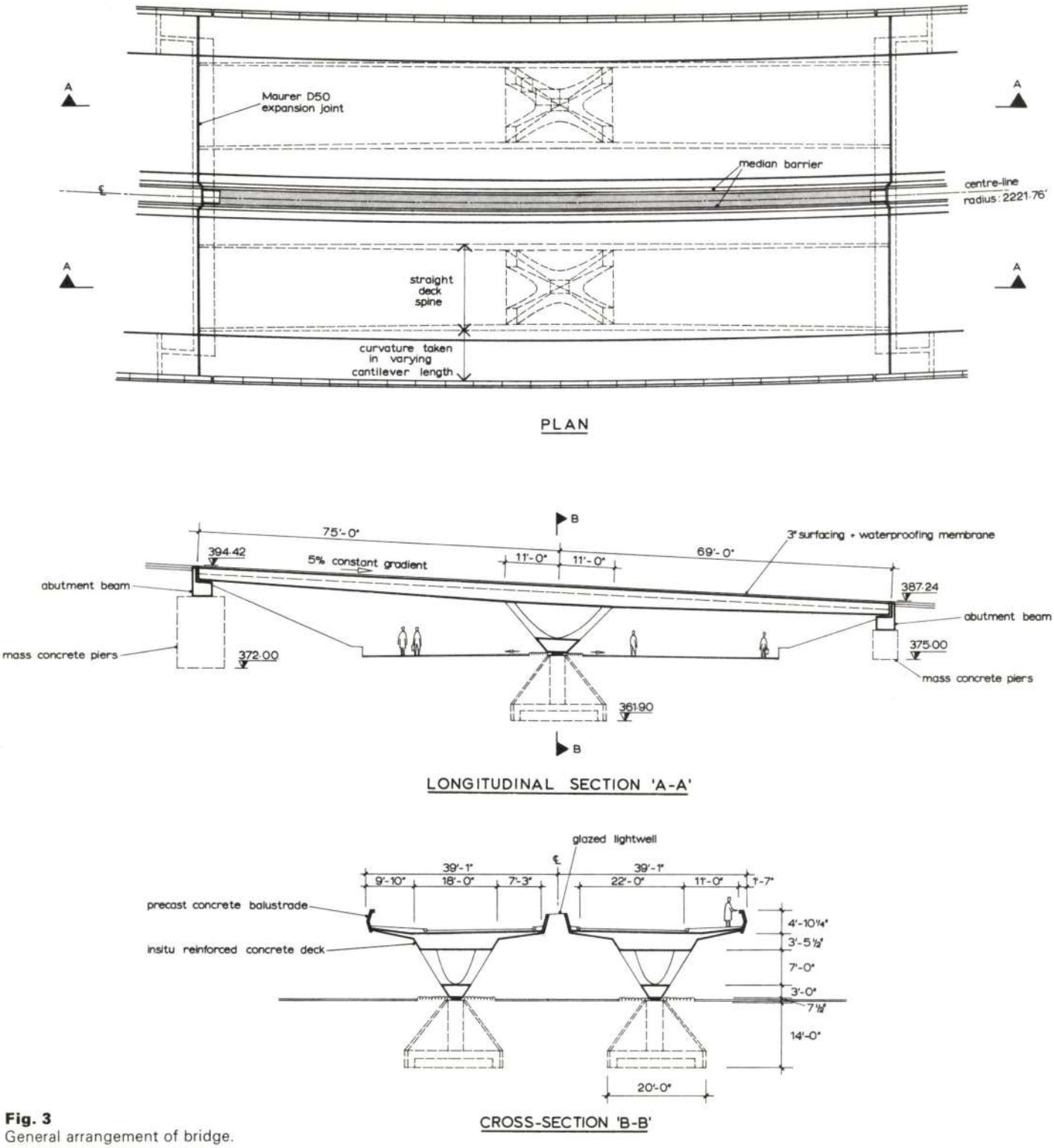


Fig. 3
General arrangement of bridge.

not helpful, there being twice the seasonal rainfall. However, by the end of October the construction of a long 3 ft. 6 in. (1.1 m) sewer had been completed together with the following work:

1 50% of bulk excavation.
2 The central support foundation to the south bridge.
3 Partial completion of the abutment mass foundations.
4 The services diversions to the south of Western Bank.

The excavation for the south bridge was then completed, and erection of the falsework for the deck commenced from each abutment, concurrently with the erection of the central support formwork.

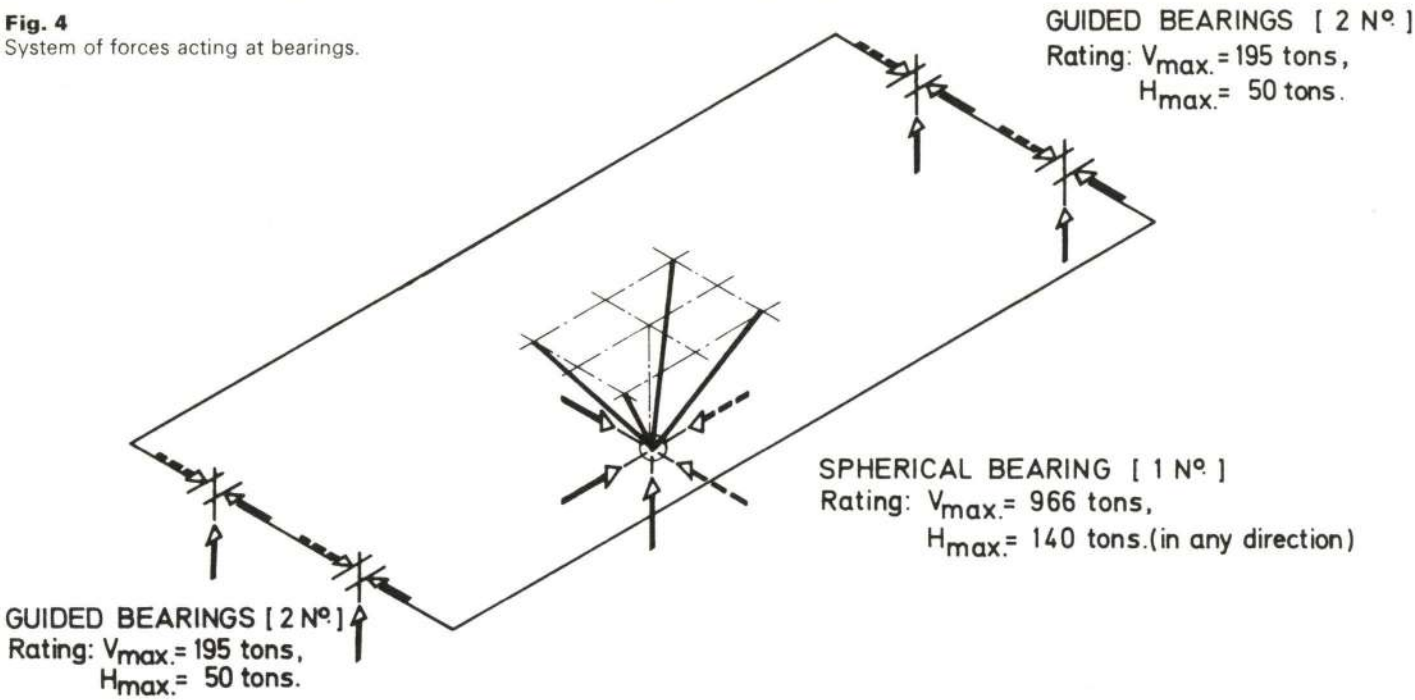
The latter formwork had been previously assembled to check that the components fitted correctly, as the geometry was quite complex. Plastic faced ply sheets with timber and steel backing frames were used for this formwork.

The central support was to be poured in a single operation and any movement during construction could create considerable difficulties. The outside shutters were therefore supported very firmly and the core top shutter anchored down to the foundations against the several tons of uplift that would occur. Ready-mixed concrete was used throughout the works and this created problems, as the only access was from the open half of the dual carriageway. The carriageway was therefore closed on a Sunday morning. The concrete was first poured through a central 9 in. (230 mm) diameter tube into the base until it rose up into the bottoms of the legs and then down each leg simultaneously. The result achieved, considering the difficulty of the operation, was quite good. Credit should go to the Ready-Mix supplier for his co-operation in producing a mix with the necessary strength and workability.

The bridge deck was then concreted in a continuous pour of 414 cu. yds. (316 m³) at the contractor's request. Concrete was placed from the centre towards each abutment through Mark-Thomson lorry mounted pumps. They were each capable of placing up to 40 cu. yds. (30.6 m³)/hour. The weather was perfect. Work started shortly before dawn and all went well until both pumps were blocked. It was discovered that one lorry load of aggregate had been supplied to the plant at ¾ in. (20 mm) single size instead of ¾ in. (20 mm) and smaller. Over 60 cu. yds. (46 m³) of concrete either on site or in transit had to be wasted. The pumps were cleared without too much trouble and pouring was completed in 8½ hours. Care had to be taken throughout to keep the long 'wet edge' of concrete moving to avoid pour lines.

We now had a rather traumatic experience prior to the unpropping of the deck, when we

Fig. 4
System of forces acting at bearings.



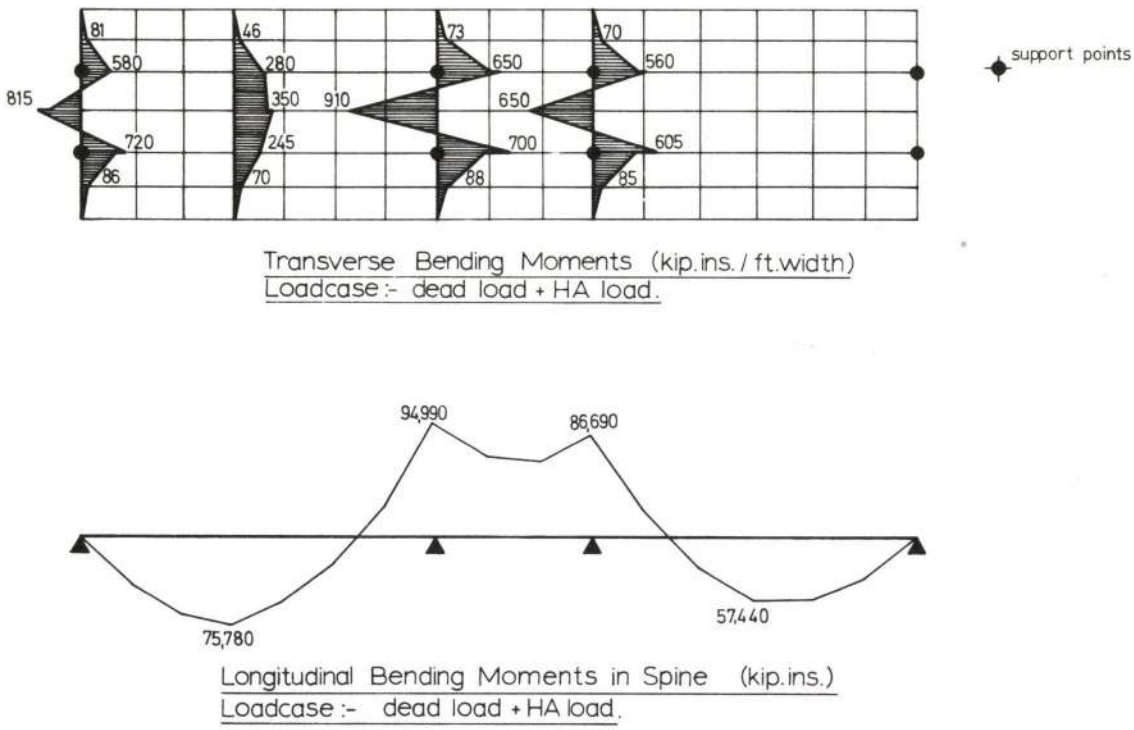
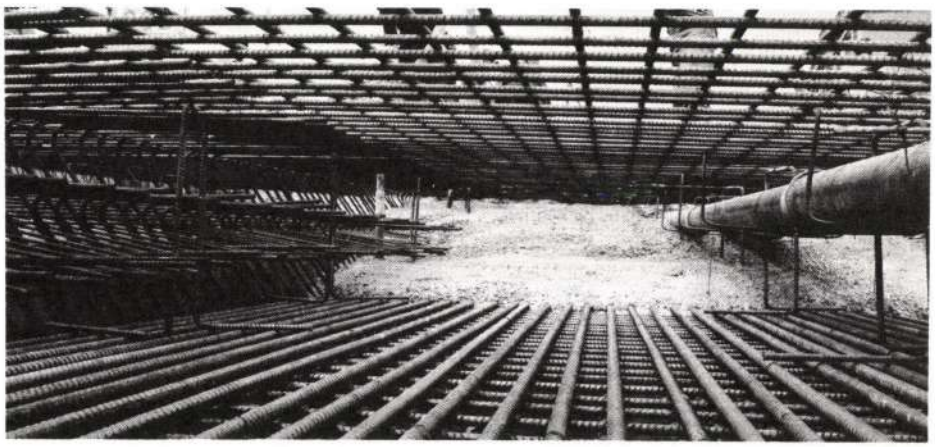


Fig. 5
Typical distribution of bending moments.

discovered that the temporary restraining straps on the central support bearing had sheared off. Movement of the whole support had taken place. It transpired that the bearing as delivered to site, did not have the main bearing faces in contact and that the straps had sheared at some stage prior to, or at the very beginning of, the deck pour. Fortunately the movement which had taken place still left sufficient rotational capacity in the bearing for it to function properly. We obtained a new unconditional guarantee from the manufacturer who modified the assembly process of this type of bearing as a result of this mishap.

The south bridge was opened to traffic on 23 March and work commenced on the north bridge, ten weeks behind the original programme. The weather over the winter months had not been helpful either, there being over 3 ft. 4 in. (1 m) of total snowfall. However, with no services to divert and most of the problems solved, the second deck was

Fig. 6
Long wet edge of concrete.
(Photo: M. T. Walters and Associates Ltd.)



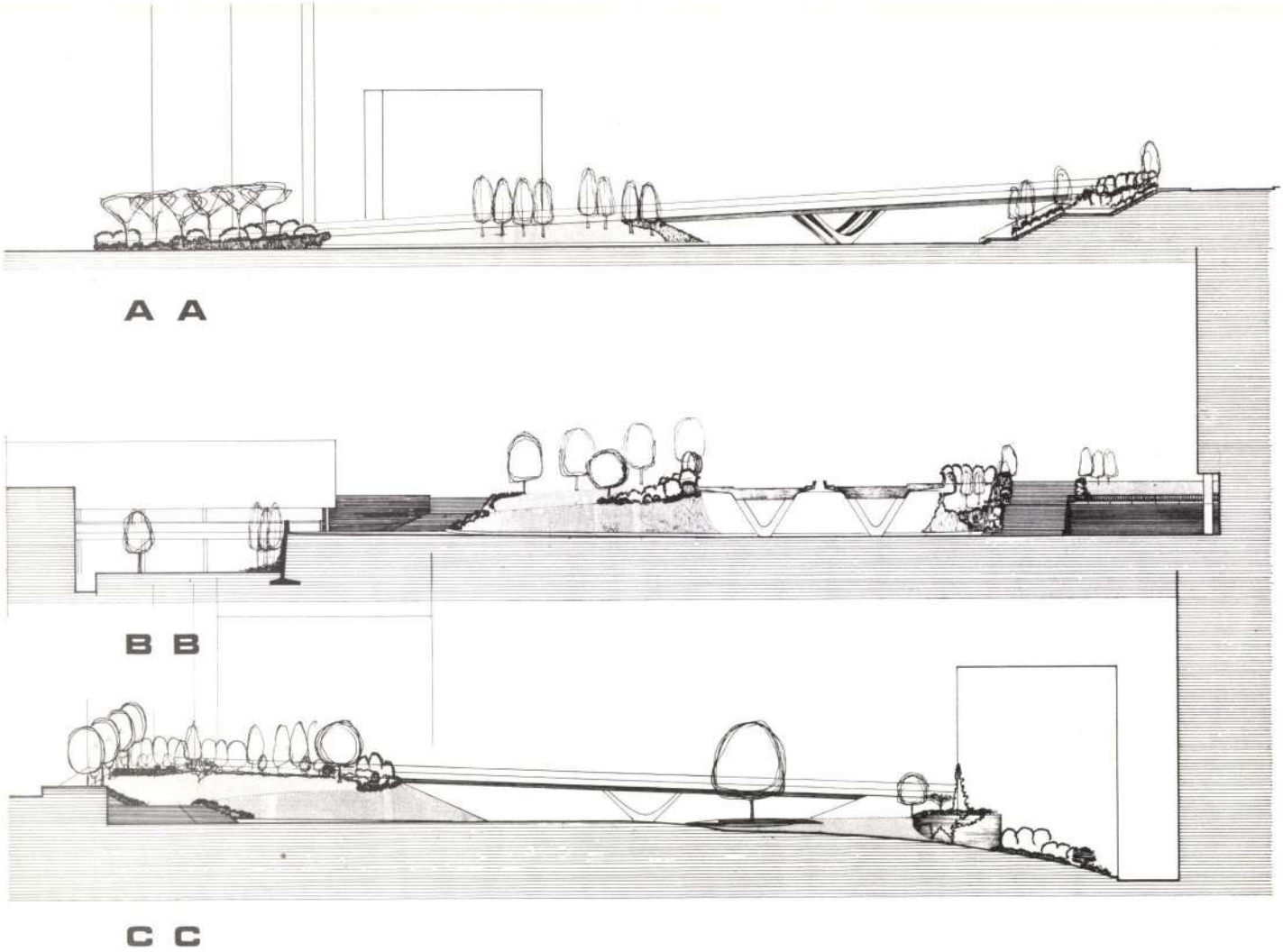


Fig. 7
Sections through concourse.

poured on 1 June, just ten weeks later. Dual carriageway working was re-introduced on 8 August and the concourse was opened on 12 September.

In view of the disruption of pedestrian routes and other disturbances, some trouble had been expected from the students during the contract. This did not really materialize though the large numbers of mini-skirted girls cost the contractor many hours of standing time. However, on the eve of May day, vandals struck with, 'STIKE (*sic*) FOR ACADEMIC FREEDOM', spray painted in 3 ft. high red lettering along the precast concrete balustrades. The message was eventually erased with acetone and water but we felt that somehow the christening had been effective.

Analysis of costs as built

Total Cost = £157,000 approximately.

The distribution of expenditure including preliminaries in due proportion is as follows:

		%
1	Earthworks	6
2	Services' diversions, electrics and drainage	13
3	Bridgeworks (sub-and superstructure)	36
4	Other structures	6
5	Roadworks	10
6	Landscaping (including paving)	29
		100%

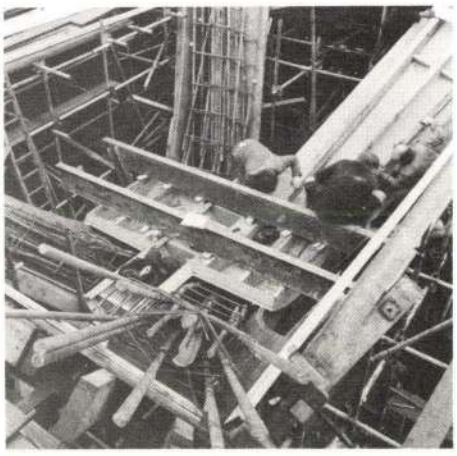


Fig. 8 above
Positioning of top shutter to central support.
(Photo: M. T. Walters and Associates Ltd.)

Fig. 9 right
View from lower court. Station for large tree in right foreground.
(Photo: M. T. Walters and Associates Ltd.)

Fig. 10 left
Informal seating.
(Photo: M. T. Walters
and Associates Ltd.)

Fig. 11 below
View from Students' Union.
(Photo: Henk Snoek)

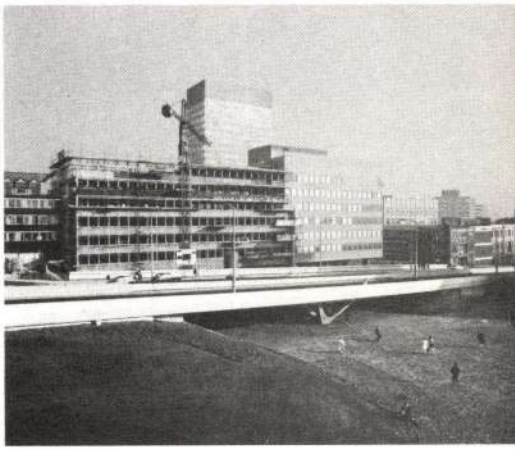


Fig. 12 above
North bridge before concreting.
(Photo: M. T. Walters
and Associates Ltd.)

Fig. 13 above
'The connecting element should have a feeling of lightness and space.'
(Photo: Henk Snoek)

Fig. 14 right
View of south concourse.
(Photo: Henk Snoek)

Fig. 15 below
Access to south concourse from Hounsfield Road.
(Photo: Henk Snoek)

Preston Bus Station and Car Park

Ralph Stephenson

Introduction

Preston is the nucleus of the proposed Preston-Chorley-Leyland new town(1), often referred to as Ribbleton, which, it is hoped, will be built up during the 1970's and 80's. The M6 motorway passes close by to the east and a future spur from this motorway, the M61, will connect Preston directly to Bolton and Manchester. The M62 Lancashire/Yorkshire motorway, now under construction, will cross the M6 and M61 to the south of Preston. North-east Lancashire, which is in danger of becoming a depressed area, will be connected by a link road to the new town.

The bus station is integrated with the new road system of central Preston and it will accommodate more buses than the total using the conglomeration of small congested stations of the separate companies operating at present.

Early history

The architect was appointed by Preston Corporation in 1959 to produce a scheme for a bus station and separate park for 500 cars. Since then there have been many changes.

The name of the architect has changed from Grenfell Bains and Hargreave to Design Partnership and subsequently to Building Design Partnership and the scheme has changed several times from the one described above. The second scheme became a bus station concourse with office accommodation. This, in its turn, evolved into a bus station with a separate multi-storey car park, circular in plan. This subsequently became a scheme for a bus station with a wholesale fruit and vegetable market overhead. The latter finally changed to the present scheme for a bus station for 80 buses, concourse, office accommodation and car park overhead for 1,100 vehicles, the brief for which was presented to the architect in January 1965.

The estimated overall cost, including elevated ramps, subways and external works produced by the quantity surveyor (also Building Design Partnership) in April 1965, was £1,062,000.

General description

The bus station concourse, which provides stands for 80 buses, waiting rooms, small shops and offices and canteens for the bus companies, measures 100 ft. x 560 ft. (30 m x 171 m) and is placed centrally on the 314 ft. x 616 ft. (96 m x 188 m) site. Down each side of the concourse are the bus parking and circulation areas, and overhead are the car parking floors which overhang the concourse (Figs. 1–5).

The bus circulation area is covered with 9 in. (230 mm) thick reinforced concrete paving with a carefully arranged pattern of expansion and contraction joints to match the bus parking bays and the building module.

Access to the concourse is via one of three subways, each subway leading to either an existing or a planned shopping development; segregation of pedestrians and traffic at ground level being ensured by the use of large boulders in the landscaping and shoulder height precast concrete units forming barriers. The concourse is fully glazed on all sides, the glazing of the long sides being in the form of 13 ft. 4 in. (4 m) wide sliding doors at each bus stand (Fig. 5). The edge

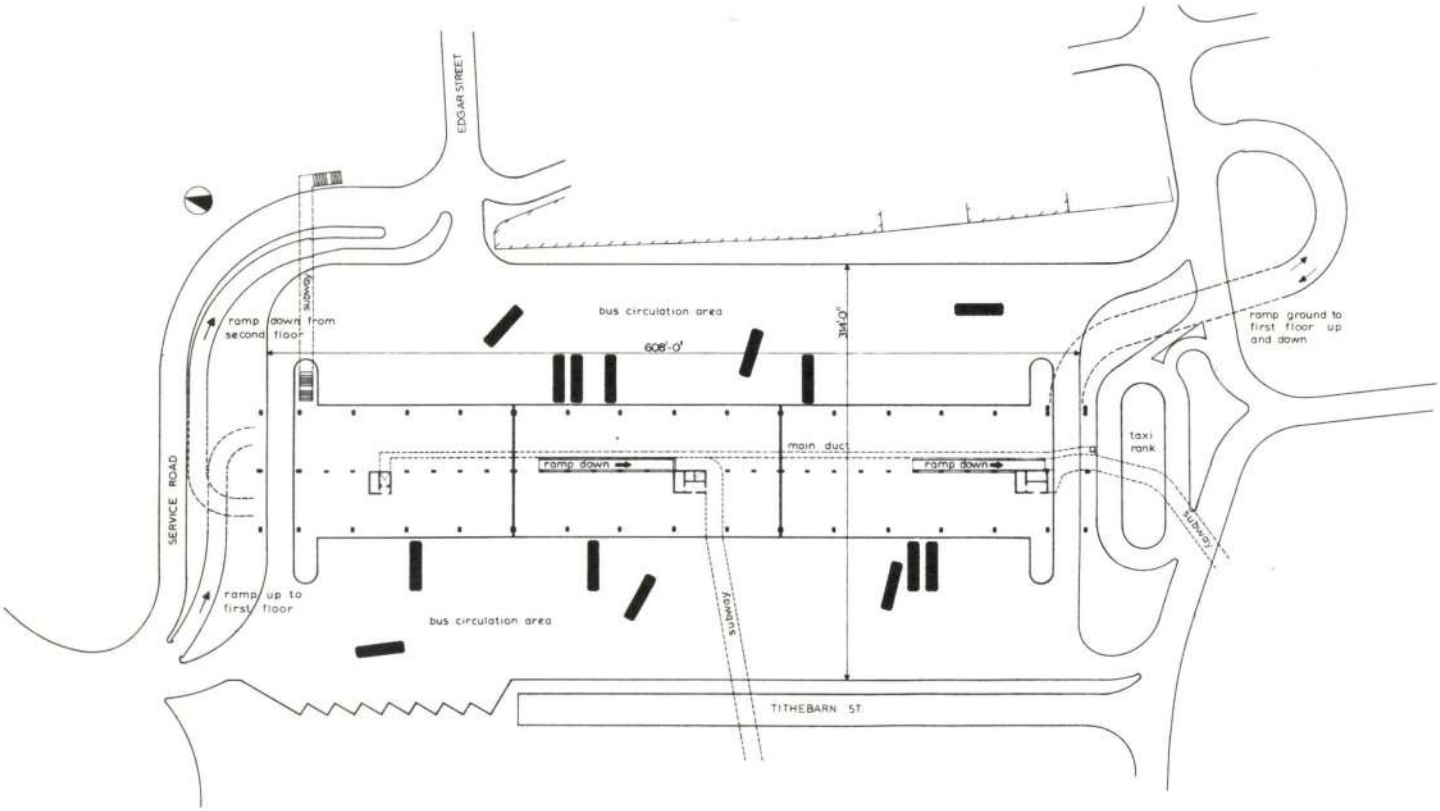


Fig. 1
Ground floor plan showing bus circulation, concourse, subways and ramps.

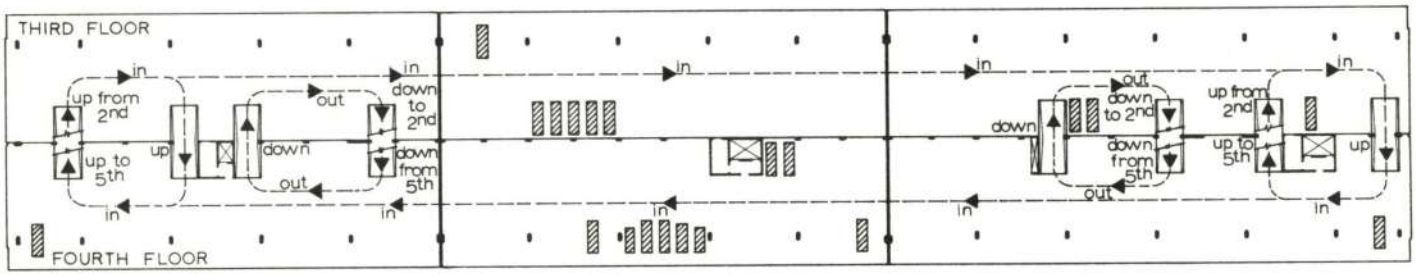


Fig. 2
Upper floor plan showing internal ramps.

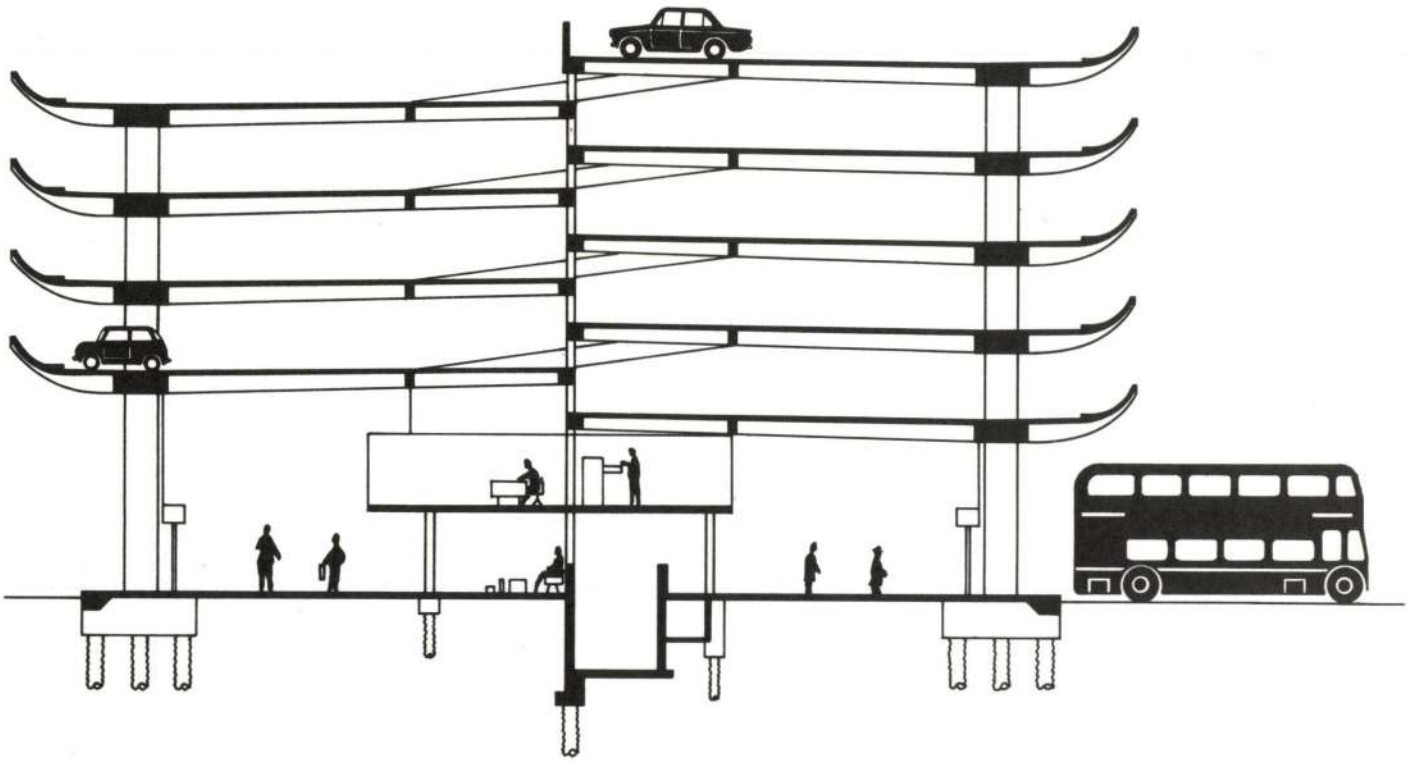


Fig. 3
Section through building.

columns which are at 40 ft. (12.2 m) centres, are 3 ft. 6 in. x 1 ft. 9 in. (1.06 m x 0.53 m) overall with 10½ in. (267 mm) radius ends. The finish to these columns is produced by grit blasting to expose the uncrushed gravel aggregate. Elsewhere in the concourse the wall and column surfaces are tiled in white and the floor is finished with ribbed rubber tiles.

Access for cars to the parking floors is by separate entrance and exit ramps at the northern end, and by a single combined ramp at the southern end. Internally there are eight ramps which allow one way traffic either up or down.

We decided on a ribbed floor construction of precast units at an early stage in the scheming and, because the structure is expressed architecturally and so demands a consistently high standard of surface finish, we specified that the units should be cast in fibre glass moulds. The floor falls 6 in. (150 mm) to the external columns to provide drainage and, in addition, the rib depth increases from the building centre line (Fig. 3). This increase in depth was an architectural requirement to give a feeling of strength at the beam over the external columns and to provide a pronounced modelling to the soffit of the floor that cantilevers beyond the beam to form the floor balustrade. The ribs reduce in depth as they follow the cantilever's curve upward, finally petering out about 1 ft. 3 in. (380 mm) below the top of the curve and so leaving a plain un-modelled band that ties the edge in visually in a horizontal direction (Fig. 4).

Tendering

Procedure

The client, who was anxious to obtain the cheapest building that would meet his specification, insisted that we advertise nationally for 'contractors and industrialized system builders' interested in tendering. From the replies, a short list of 29 contractors was prepared and in October 1966 they were sent a set of structural drawings showing the BDP/Arup scheme with precast ribbed floor, and a performance specification in case they were prepared to put forward an alternative scheme of their own. They were asked to state which of the following methods of tendering they were interested in and, in the event of their preferring to tender on their own design, they were asked to forward drawings describing their proposals. The methods of tendering and the breakdown of replies are listed below and it can be seen that we were left with the possibility of having 40 tenders, even though seven contractors had withdrawn from the list in the meantime.

Four alternative designs were put forward. Two were in steel and did not comply with the fire regulations then in force ([2]), nor were they aesthetically acceptable to the architect. A third was a precast concrete scheme that did not comply with the performance specification. The serious competition came from the fourth scheme. This was submitted by British Lift Slab Ltd. who put forward what they describe as the 'warped slab' system for the car parking floors (Fig. 6).

Their warped floor was a 10 in. (250 mm) thick prestressed slab of lightweight concrete on a 40 ft. x 40 ft. (12.2 m x 12.2 m) column grid. The outer longitudinal edges were level and the floor, although giving split level parking, was continuous in a transverse direction for 50% of the length of the building. We went to Birmingham to see the Co-op Dairy milk float park that had floors built to this system, though with much smaller spans and normal weight concrete, and David Ascough of British Lift Slab Ltd. drove us round it at a great rate of knots to demonstrate that the warp held no fears for drivers. After giving the scheme a lot of consideration BDP advised the client that it should not go forward as a competitor for the following reasons:

1 There were adverse cambers at the connections between the two halves of the building and at the north end ramps.
2 There were difficulties with the cladding of the mezzanine floor and the ends of the car park.
3 The floors had poor quality soffit finishes.
4 The 'slow' curve of the floors was not aesthetically pleasing.
5 Double columns on the centre line did not suit the office and the concourse planning.
6 The falls of the floors produced drainage difficulties.

In December 1966, the QS advised the client that, as the contractors had shown a marked preference for our scheme and as no acceptable alternative to it had been produced, we should select ten contractors from those remaining, the maximum number recommended at that time by the Code of Procedure for Selective Tendering being 12. The client agreed and in June 1967 a selection of contractors, medium and large, local and national, were invited to tender.

Table 1 Analysis of tenders

Tender		Number of contractors interested	
1	A bill of quantities for the work as shown on our drawings		15
2	A bill of quantities prepared from the contractor's own design	Our design but in situ Alternative design	5 3
3	As a nominated sub-contractor for the structure only in 1		4
4	As a nominated sub-contractor for the structure only in 2	Our design but in situ Alternative design	3 4
5	As 1 but the structure being executed by a nominated sub-contractor		6

Costs

The tenders were opened on 23 August 1967 and the lowest was that of John Laing Construction Ltd. at £1,213,670. The second tender figure was £1,262,810 and the highest was £1,562,000.

The office accommodation worked out at 185/– per sq.ft., the bus station concourse at 94/3 per sq.ft. and the car park at 35/6 per sq.ft., although it must be said that it was difficult to apportion the cost between concourse and car park.

The drainage and external works, including the ramps, amounted to £260,000 and the preliminaries, insurances, etc., to £195,000.

After studying the QS report, the council committee of the Borough of Preston said that the cost must be cut by 20% but they recognized that such a cut could only be made by sacrificing some of the facilities. The saving achieved was £190,000, less than 20% but acceptable to the committee, the bulk of the saving being due to the removal of public restaurant and recreational facilities, ramp heating, lifts and landscaping. Nothing was taken out of the structure.

On 18 November 1967 the pound was devalued and, on 20 November, Laing's withdrew their tender, but expressed a willingness to negotiate a new figure, taking into account the new reduced value of the pound and the increased Bank Rate. This was done and, in January 1968, a new tender figure was agreed at £1,082,720.

Foundations

Ground conditions

The Geological Survey of Great Britain shows that central Preston has up to 73 ft. (22.3 m) of sand and gravel resting on Bunter sandstone. The site investigation, carried out in 1966 by Geo Research, shows that the site is overlain with 4 ft.–7 ft. (1.2 m–2.1 m) of fill. Below this is fine brown sand with thin bands of silt in its lower levels down to about 30 ft. (9.1 m) where there is a thin layer of clay. Below the clay is compact sand which becomes dense at depths of 37ft.–45ft. (11.3m –13.7 m). (See borehole C in Fig. 7.) No water was encountered.

With column loads in the 600 to 800 ton region, end bearing piles in the dense sand were decided upon.

Tenders for installation of piles

In July 1967, nine piling contractors were invited to tender, alternative quotations were invited for either, or both, driven or bored piles. Consideration of the ground conditions above favoured driven piles but some of the adjacent property was very dilapidated and the tenderers' attention was drawn to this. They were asked to found in the dense sand.

Only four tenders were received. Three were for bored piling, all of which were considerably more expensive than the single driven piling tender. The reasons for the drop-outs were:

1 Firms specializing in driven piling who thought that bored piling was the answer.
2 Firms specializing in bored piling who thought that driven piling was the answer.
3 Firms who thought that their equipment would not install the capacities of piles that we were looking for.

The lowest tender received was from GKN Foundations Ltd. at £41,565. For this figure they would install 636 17 in. (430 mm) diameter Holmpress re-driven piles of 65 ton capacity to an average depth of 41 ft. (12.5 m) and they would carry out six working pile tests. Preliminary tests were extra and included in a separate quotation.

The lowest bored piling tender figure was £47,268. The tenderer proposed using 24 in.–48 in. (610 mm–1.2 m) diameter straight shafted piles taken 1½ diameters into the dense sand. Again, preliminary tests were extra.

Fig. 4
The bus station looking from the south.
(Photo: Building Design Partnership)

Fig. 5
Inside the concourse. Perspective drawn by Alan Mitchell.
(Photo: Building Design Partnership)

Table 2 Driving record of test piles

Depth in feet	No. of blows per foot required to drive piles.	
	17 in. test pile	20 in. test pile
27	12	18
28	10	14
29	9	12
30	8	12
31	10	15
32	12	17
33	10	20
34	10	20
35	11	23
36	13	25
37	19	24
38	21	26

Preliminary test piles

During discussions with GKN, they expressed confidence that they could found their 65 ton piles at a much higher level than the specification called for and that this would show a saving on their tender figure. We, however, were keen to increase the pile capacity by driving to the greater depth, and, by using a 20 in. (510 mm) diameter pile, this would show a saving in the pilecaps and the time taken to install the piles. In addition, we were convinced that the cost of pile per ton carried would be reduced. From Meyerhof's papers ([3 4 5]) we estimated the ultimate capacity of the 17 in. (430 mm) and 20 in. (510 mm) piles driven to the dense sand at its maximum height of 37 ft. (11.3 m) to be:

	17 in. diameter	20 in. diameter
Meyerhof 1953	236 tons	326 tons
Meyerhof 1956	248 tons	346 tons
Meyerhof 1959	273 tons	378 tons

Two test piles were installed as near to a borehole as could be managed. One pile was 17 in. (430 mm) diameter with a proposed working load of 65 tons, the other was 20 in. (510 mm) diameter with a proposed working load of 90 tons and both piles were driven by means of a 2½ ton hammer falling 6 ft. (1.8 m). Despite GKN's insistence that the 65 ton pile could be founded at a high level, they drove both piles to 38 ft. (11.6 m) i.e. to the dense sand. The number of blows per foot required to drive the piles through the last 12 ft. (3.7 m) are shown left in Table 2.

The piles were loaded by jacking off anchor piles and the load was applied in accordance with our old piling specification; the first cycle to 1.5 x working load in increments of 0.25 x working load, and the second cycle to 3 x working load, again in increments of 0.25 x working load. The maximum loads in each cycle were maintained for at least 24 hours and the intermediate loads for 1 hour.

The load/settlement figures obtained were:

For the 20 in. (510 mm) 90 ton pile with a load of 315 tons applied (3.5 x working).

Settlement at application of last increment 0.33 in. (8.4 mm)
Settlement after 24 hours 0.38 in. (9.7 mm)
Settlement on removal of last increment 0.16 in. (4.1 mm)

For the 17 in. (430 mm) 65 ton pile with a load of 225 tons applied, the corresponding figures were 0.33 in. (8.4 mm), 0.4 in (10.2 mm), and 0.22 in. (5.6 mm).

The specified failure load was the load at which the pile settlement exceeded 1/10 of its base diameter. The above results were therefore very encouraging. The load/settlement and recovery curves were almost straight, the rate of settlement under 3.5 working load being 0.0021 in. (0.05 mm) per hour for the 20 in. pile.

After studying the results of the test piles, GKN agreed to increase the working load on the 20 in. (510 mm) pile to 100 tons, giving a stress of 715 lbs/sq.in. (4.9 N/mm^2) on the concrete. The capacity of the 17 in. (430 mm) pile could also have been increased but, as these are now used to support only the lightly loaded columns, and often singly with quite a lot of capacity to spare, the need did not arise.

A revised tender figure was agreed at £38,142 for 130 65 ton piles and 370 100 ton piles and a rate per pile was fixed at £44 10s. for the 65 ton capacity and £58 10s. for the 100 ton capacity. The time required to install the piles was reduced from 14 weeks to 11 weeks. The cost of the test piling was £2,260.

Further site investigation

Before commencing the installation of the working piles, GKN did five boreholes of their own for their own use. Two of these, no. 1 and no. 2, were adjacent to the test pile set up, no. 1 being about 20 ft. (6.1 m) away from it and no. 2 being between the test piles. GKN borehole no. 5 was close to Geo Research borehole E. The logs of these holes, and SPT (Standard Penetration Test) values obtained from them, are shown in Fig. 7.

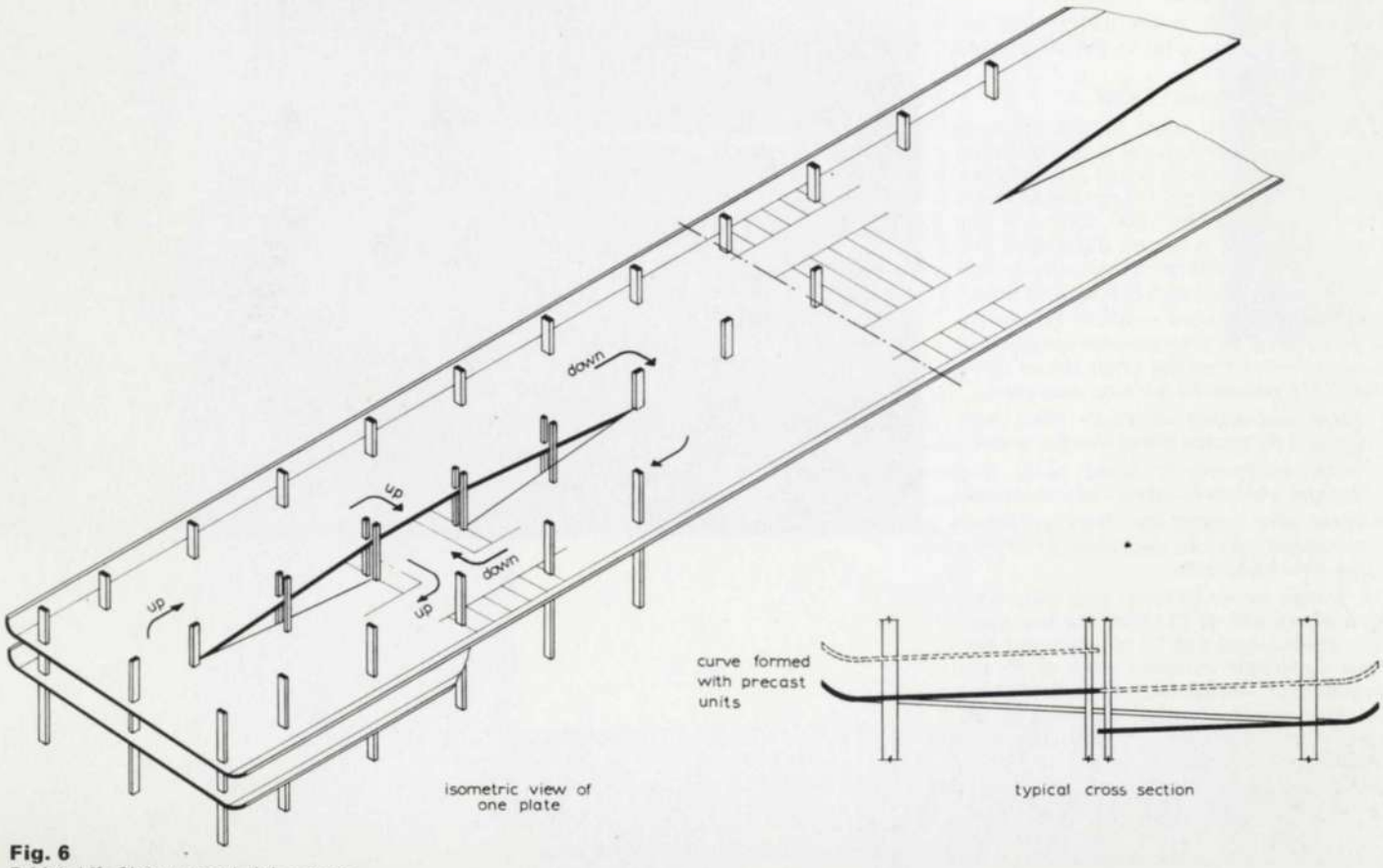
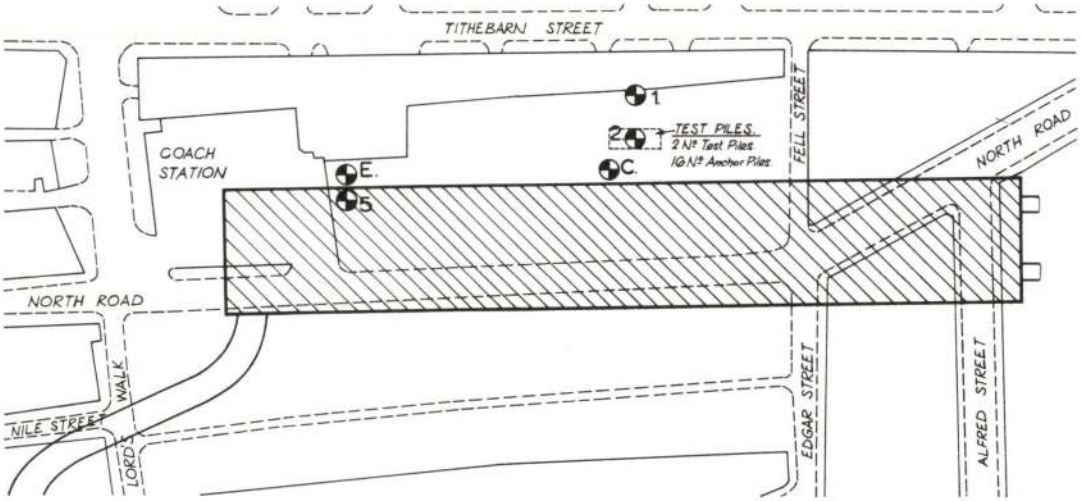


Fig. 6
British Lift Slab warped slab system.



BOREHOLE 'C'

STRATA	DEPTH	N
	G.L.	
Cinder & brick fill	4′ 0″	
Light brown fine sand with thin bands of silt & gravel		10
		8
	26′ 6″	7
Loose light brown silt	33′ 0″	2
Firm brown clay.	34′ 0″	
Compact light brown sand.	37′ 6″	23
Dense fine light brown sand	46′ 6″	40
End of borehole		40

BOREHOLE '1'

STRATA	DEPTH	N
	G.L.	
Made ground— ashes, brick, rubble, sand.	2′ 6″	
Loose fine silty sand		4
Moderately compact reddish brown medium grained sand with occasional fine gravel	7′ 0″	16
		17
		13
		23
Fine silty sand.	29′ 0″	13
Moderately compact to compact reddish brown fine sand	31′ 0″	17
		21
		34
		22
Very compact fine reddish brown sand	47′ 0″	80
End of borehole	50′ 0″	50

BOREHOLE '2'

STRATA	DEPTH	N
	G.L.	
Made ground— sand, brick fragments, rubble	5′ 0″	25
Moderately compact mixed reddish brown sand with occasional fine gravel.		32
Compact fine red silty sand	9′ 0″	
		28
		34
		36
		37
		39
Very compact mixed brown sand	40′ 0″	56
		60
End of borehole	50′ 0″	50

BOREHOLE 'E'

STRATA	DEPTH	N
	G.L.	
Brick clay & cinder fill	4′ 0″	
Compact light brown sand		9
	23′ 0″	7
Loose light brown sand with bands of soft brown clay		2
		3
	38′ 0″	2
Dense light brown sand		41
		42
End of borehole	51′ 6″	46

BOREHOLE '5'

STRATA	DEPTH	N
	G.L.	
Made ground ashes, brick, rubble, sand.	1′ 6″	
Dark grey silty sand.	3′ 6″	
Moderately compact mixed brown sand with occasional gravel		13
		14
		14
		13
Loose fine brown silty sand	21′ 0″	9
		8
		11
Compact becoming very compact with depth. Fine to medium brown sand	36′ 0″	29
		35
		50
End of borehole	50′ 0″	63

Boreholes 'C' & 'E' by Geo-Research Ltd. 1966. Boreholes '1', '2' & '5' by G.K.N. Foundations Ltd. after test piles installed.

Fig. 7
Borehole information.

Working piles

Installation of the working piles commenced on 11 March 1968. From the sets of the 20 in. (510 mm) test piles it was agreed that the working piles should be driven 6 ft. (1.8 m) beyond the level at which a resistance of 20 blows per foot was encountered, and that over this 6 ft. (1.8 m) the sets should be 20 or more blows per foot. A further stipulation was that the above blows should be recorded after penetration of the clay layer.

We had a few difficulties, the major one concerning a five pile group in sand looser than that encountered elsewhere on the site. The piles were driven to the maximum depth possible with the equipment available, without encountering the dense sand. GKN insisted that the settlement of these piles would be within our specification requirement and that the differential settlement between the suspect group and adjacent groups would not be such as to cause any distress to the structure. During our discussions two important points came to light. One was that GKN considered their pile to be a friction rather than an end bearing pile, their calculated ultimate load for a 40 ft. (12.2 m) 20 in. (510 mm) diameter pile in loose sand being made up of 273 tons friction and 176 tons end bearing. The other point was that, as the agreed blows counts to which they were supposed to drive were not written into the contract documents, and as they considered that the piles were satisfactory, the contract would have to bear the cost of any additional piles called for.

A borehole was put down adjacent to the group and this showed loose sand to 60 ft. (18.3 m) then 18 in. (460 mm) of stiff brown clay followed by dense sand. Our calculations, based on Meyerhof, showed that, at ultimate load, the friction would be 44 tons and the end bearing 234 tons which indicated that, as far as load bearing capacity was concerned, an extra two piles were called for. However, in order to limit the differential settlement between this group and the others founded in the dense sand, we instructed that four piles should be added to the original five.

The parking floors

Precast units

There are five precast unit types, A, B, C, D and E, and details of these are shown in Fig. 8 and Table 3. Each type has variations to accommodate services, fixings and reinforcements, but none of the variations affect the basic moulds.

The final shape of the units was fixed after discussing possible demoulding problems with the contractor, who thought that our original steeper sided rib (5°) would not strip out of the mould. He also thought that the stiffening rib projection and the end faces would lock in and so he suggested a mould that could be split into several pieces to avoid these problems. We, however, wanted a one piece mould that would give a unit unmarred by blemishes at shutter joints and so, to alleviate the contractor's anxiety, we increased the splay of the rib and the radii of the corner curves.

The timber formers for the moulds were made in Blackpool by Messrs. Glasdon Signs Ltd. and the fibre glass moulds were made in Nelson by Bennett Plastics Ltd. Thirty moulds were made in all, 12 of type A, 1 each of nos. B, C and D and 15 of type E, and these were then set up in the site casting yard by the contractor (Fig. 9).

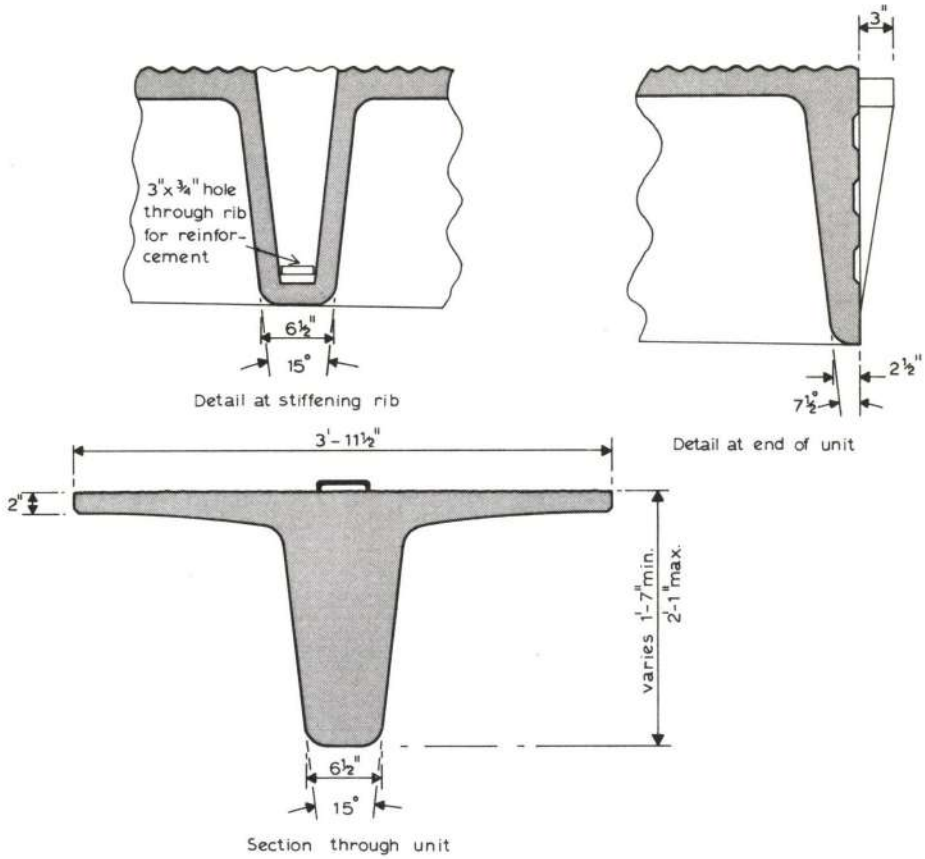
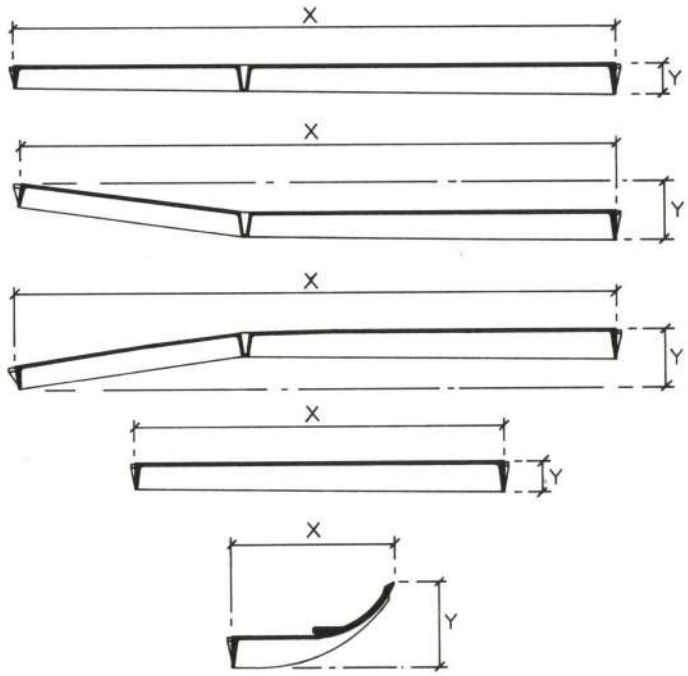


Fig. 8 right
Details of precast units.

Table 2 Typical details of precast units.



Type	No. off	Weight (tons)	Dimensions X	Dimensions Y	Cost/ unit erected
A	1132	5·75	40′-5″	2′-1″	£83
B	108	5·75	40′-1″	4′-1″	£85
C	106	5·75	40′-6″	4′-0″	£83
D	68	3·75	24′-11″	2′-1″	£50
E	1395	1·75	11′-0″	5′-9″	£27

A concrete base was laid over the casting area and to this were fixed timber cradles which support the moulds. The cradles are at 3 ft. (910 mm) centres and have a removable tie across the top to prevent the mould bowing in its length. They also have guides and stops which allow the mould to lift about 3 in. (76 mm) off the cradles with the unit when it is being stripped. This lifting of the mould was introduced by the contractor to help the stripping operation, the idea being that the flexible mould would tend to peel off when the unit was supported at its lifting points by the crane. The units are demoulded 24 hours after casting and they are then stacked between the rails of the derrick crane until they reach their designed strength and are needed on the job. In practice there have been no demoulding problems and 15 units per day are leaving the yard.

The finish to the top of the flange is produced by wire brushing the green concrete in the direction of the flange width to produce a surface suitable for keying the in situ topping. In addition to this key there are depressions in the flange edges and projecting links from the web (Fig. 9).

A concrete mixing set-up, separate from that for the in situ work, is being used. The aggregates are ½ in. (12 mm) maximum size crushed Silverdale limestone and 3/16 in. (5 mm) down sea-dredged washed gravel; mix proportions are 1 : 1.8 : 3.2 using ordinary Portland Ribblesdale cement, and a water/cement ratio of 0.55. The specified 28 day strength is 4500 lbs/sq. in. (31 N/mm²).

With this mix the units dry out to a fairly uniform light grey colour but during the first weeks of casting, some units, mainly type E, showed areas of discolouration which are not lightening significantly with the passage of time. Laing's research and development department investigated the problem and made recommendations that have resulted in improvements in the appearance of subsequently cast units. The three main discolourations caused in the casting yard are described below.

1 Fairly large areas of dark shading

This type of discolouration has been discussed in several publications ([6]) and it is often seen when impermeable shutters are externally vibrated. The pumping action of the shutter causes segregation of the mix near the surface and movement of water along the shutter face, both of which can lead to a blotchy finish. This is especially so when the amount of vibration and flexibility of the mould vary along its length.

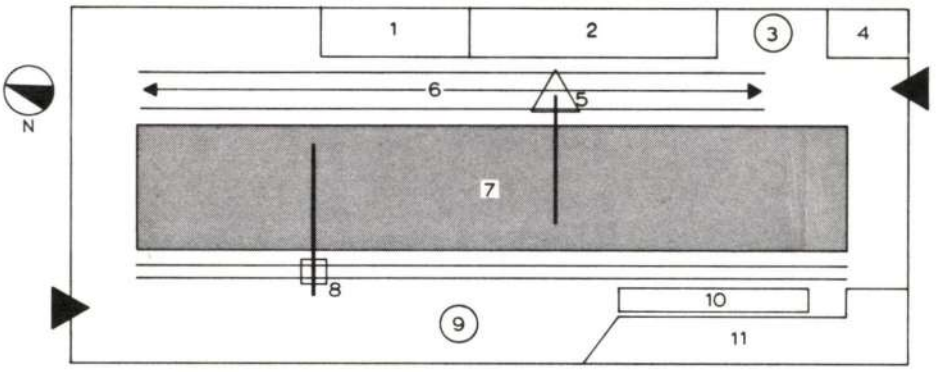
After carrying out tests on the site it was found that the amount of external vibration required could be very much reduced. It was also found that mistakes were being made with the weighing of the aggregates which were resulting in an undersanded mix. Tests showed that changes in mould oil from the original (*Nox-crete*) had no effect on the finish, but slight roughening of the highly polished mould surface with fine emery cloth tended to produce a more uniform surface. This roughening technique was not adopted.

2 Dark horizontal lines at the top of the type E unit web

These were due to the interruption of concreting when, after filling the web, concreting had to stop while the top shutter to the upward curve was fixed. Modifications to the sequence of operations were made to reduce the delay between concreting the parts of the unit and the dark lines ceased to appear.

3 Arc shaped marks on flange soffits

These were due to excessive amounts of mould oil being swabbed on with a sponge. The introduction of a sponging on, then squeegeeing off, technique, coupled with protection of the oiled mould from rain water, removed this problem.



KEY

- ◀ Access point
- 1 Steel fixers
- 2 Casting yard
- 3 Mixer for precast concrete
- 4 Joiners
- 5 Travelling derrick crane for precast concrete
- 6 Unit stacking
- 7 Building 600 ft. x 100 ft.
- 8 Travelling tower crane for in situ concrete
- 9 Mixer for in situ concrete
- 10 Contractor's offices
- 11 Existing bus station

Fig. 9
Layout of contractor's plant.

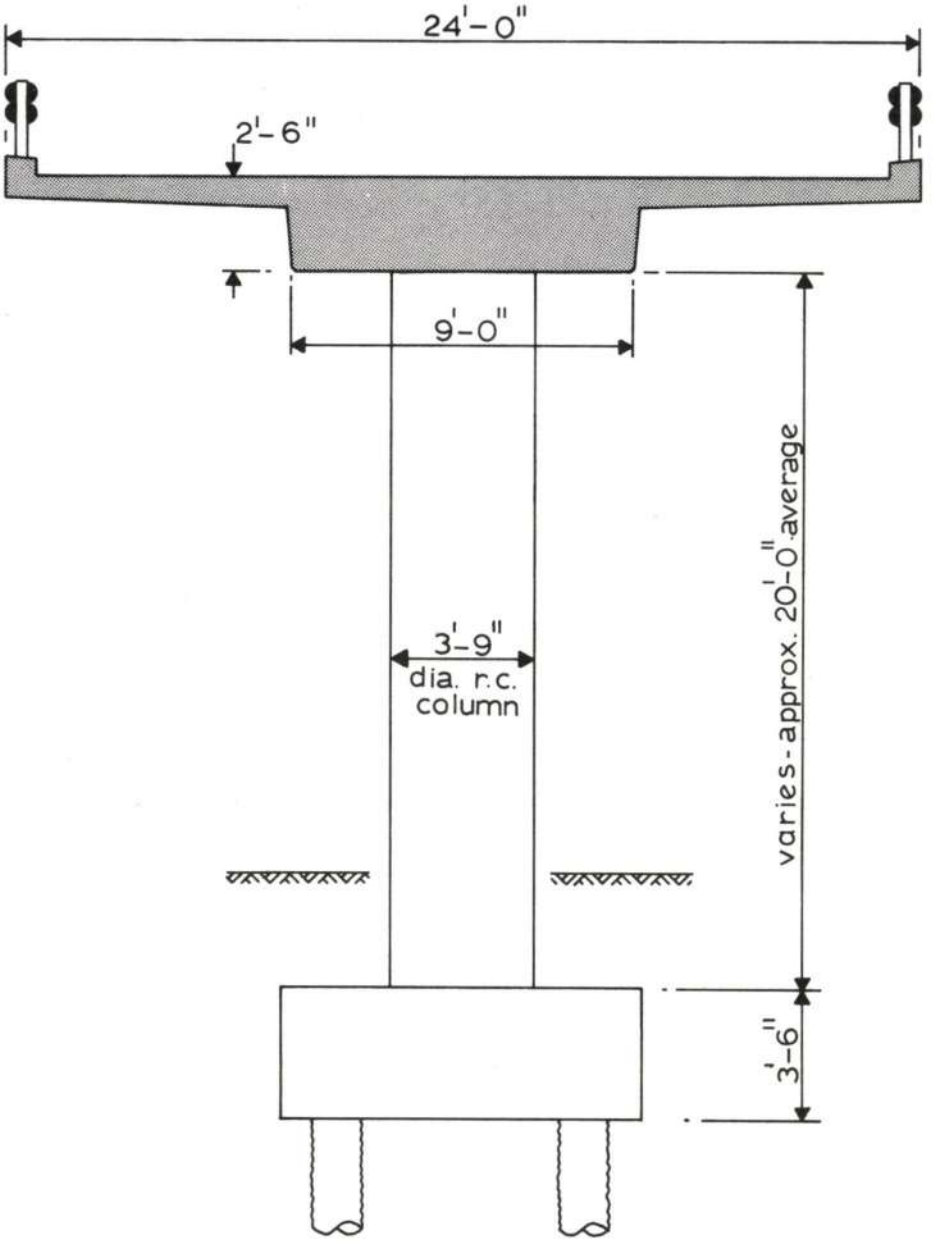


Fig. 10
Section through ramp.

Construction

The floor units are temporarily supported at their ends by the soffit shutters of the longitudinal in situ beams which, because of the weight of the construction, are in turn supported on a robust system of props. As the design super load is only 50 lbs/sq.ft. (2394 N/m²), the props have to be taken down through three storeys to spread the weight of unfinished floors over a sufficient number of beams. To avoid the need for excessively heavy propping, the contractor has worked out a system of de-propping and re-propping so that no lift of props ever carries the dead weight of more than one floor.

In an attempt to match the quality of finish of the precast unit end faces which form the sides to the longitudinal beams, the beam soffit shutters are of phenolic resin plywood with carefully placed joints.

A 2 in. (50 mm) thick mesh reinforced structural topping is laid in 12 ft. (3.7 m) wide x approximately 16 ft. (4.9 m) long bays, the finish being produced by close hand tamping with a narrow board. On the top parking levels, and the levels immediately above the concourse, the topping is finished with mastic asphalt.

Expansion joints at 200 ft. (61 m) centres are sealed with *Kork-Pak* and *Calktite* polysulphide. Some thought was given to the question of sealing, or leaving open, the ¼ in. (6 mm) nominal width joints between the type E units which, after the kerb, are not covered with topping. These joints are at 4 ft. (1.2 m) centres and a lot of sealing is involved, but it was thought prudent to seal them, again with polysulphide.

The only snag encountered so far is that the upper edges of the type E units do not lie in a horizontal line but rather follow the cambers that were built into the longitudinal beams and so give an undulating line of small amplitude to the edge. Originally a fibre glass capping was to have been fixed along this edge but this was deleted on cost grounds.

Ramps

The ramps are of in situ construction with decks supported on single rows of circular columns. The finish to the decks is fair faced concrete but the columns are grit blasted. At the southern end of the car park the ramp is 24 ft. (7.3 m) wide, allowing two way traffic. At this end, the deck spans are up to 44 ft. 6 in. (13.5 m) across 3 ft. 9 in. (1.1 m) diameter columns and the spine beam is 2 ft. 6 in. (760 mm) deep overall by 9 ft. (2.7 m) wide, with the deck slab cantilevering either side and tapering in thickness from 10 in. (250 mm) to the edge (see Fig. 10). The edge thickness is 1 ft. 1 in. (330 mm) of which 7 in. (180 mm) is taken up by the deck slab, the remaining 6 in. (150 mm) being a kerb. At the northern end the ramps are 12 ft. (3.7 m) wide with spans up to 35 ft. (10.7 m). These are of lighter construction than the northern ramp but of similar shape.

Twin *Armco* guard rails are used, the rails being fixed either side of RHS posts which are at 10 ft. (3 m) centres. We thought it better not to have a rigid fixing between the posts and the deck edge, so a rubber pad was sandwiched between the concrete and the post base plate. The whole guard rail assembly is designed to meet the requirements of the Ministry of Transport recommendations ([7]) and the pads, which are pre-drilled to accommodate the holding down bolts, are standard bridge bearing pads supplied by the Andre Rubber Co. at a cost of 1 guinea each. Clearance is left for the pad to bulge as it is squeezed when a vehicle hits the guard rail and the amount of flexure is controlled by the holding down bolts.

The whole post base arrangement is recessed into the kerb of the ramp deck and provided with a cover.

Conclusion

The contract was signed in February 1968 and the contractor moved on to the site on 18 March 1968. The programmed completion date was for 20 September 1969, and, apart from work connected with extras to the original contract, this date was met.

The building was officially opened by Lord Stokes on 22 October 1969.

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Fig. 11
Casting yard.
(Photo: Geoff Howarth)

Fig. 12
Type A unit being de-moulded.
(Photo: Geoff Howarth)

Fig. 13
Stacking yard on east side of building.
(Photo: Geoff Howarth)

Fig. 14
North-west corner,
exit ramp and tiled cladding.
(Photo: Arthur Winter)

Fig. 15
Inside the concourse
during construction.
(Photo: Arthur Winter)

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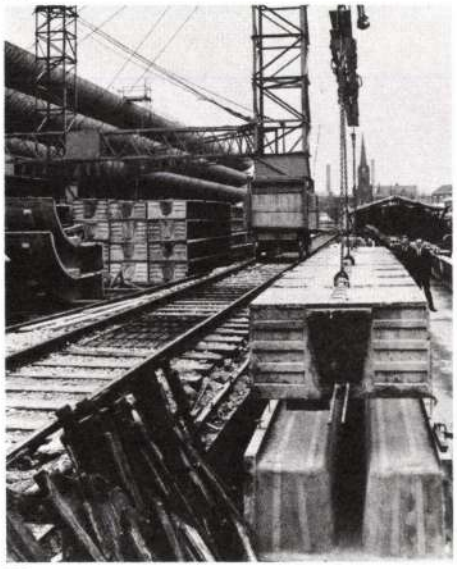

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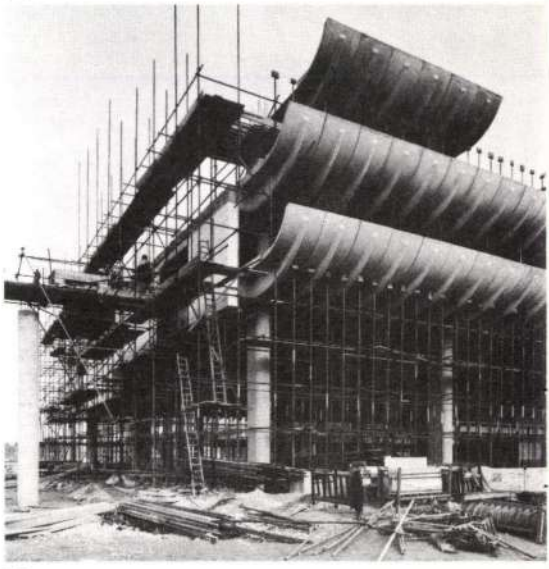

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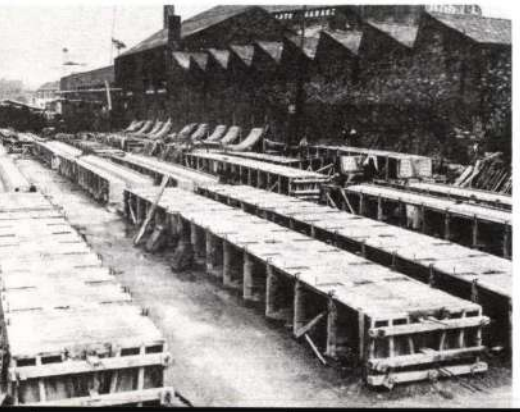

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15

Rooms for students

Tim Sturgis

Introduction

Over the last ten years, Arup Associates have designed five small residential buildings for Oxford and Cambridge colleges, and are now busy on a sixth. These buildings form a series and it is possible to follow how certain ideas have been developed by them.

Student rooms

In an article by Philip Dowson in *Architectural Design* for April 1968, the problem was defined as follows; 'The multiple function of a student bed-sitting room is a problem in opposites—a place of work and sleep, a retreat and a place to entertain, a place of privacy yet identifiably part of a larger community. How to reconcile these, and create at once somewhere with a sense of generosity and place. . .

In detail, the function of the room can be divided into four elements—a place for the bed, a desk, preferably with bookshelves and pin-up above, a storage wall usually with built-in washbasin and clothes storage, and the window wall. Rooms for graduates tend to be larger, about 200 sq. ft. (18.6 m^2), and in them there is some flexibility in the arrangement of these elements, but in the smaller undergraduate rooms, which range between 140 and 165 sq. ft. (13-15.3 m^2), little room for manoeuvre remains. However, students want to be able to express themselves and furniture arrangements should be flexible—one girl put her chest of drawers on its side in search of individuality.

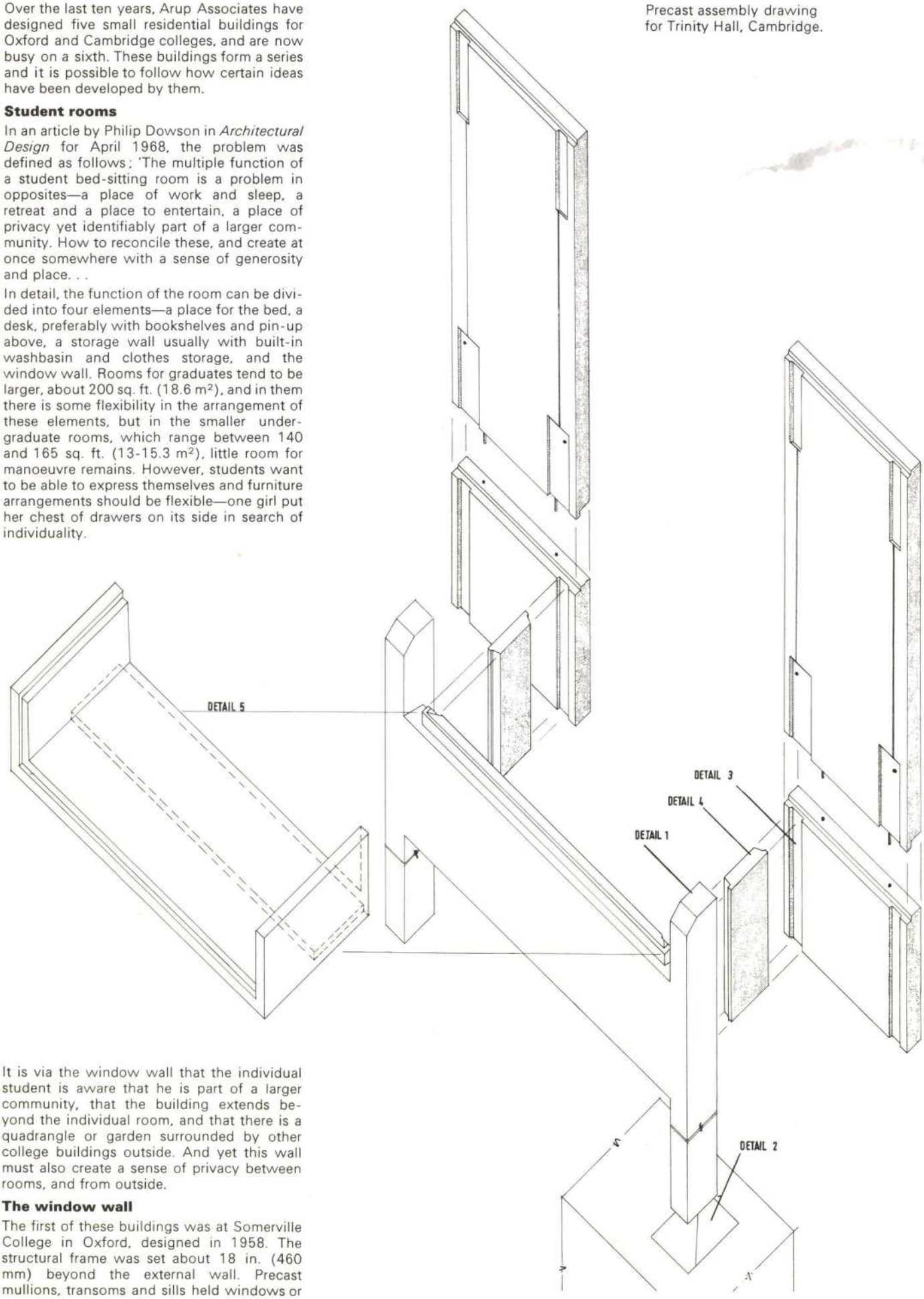


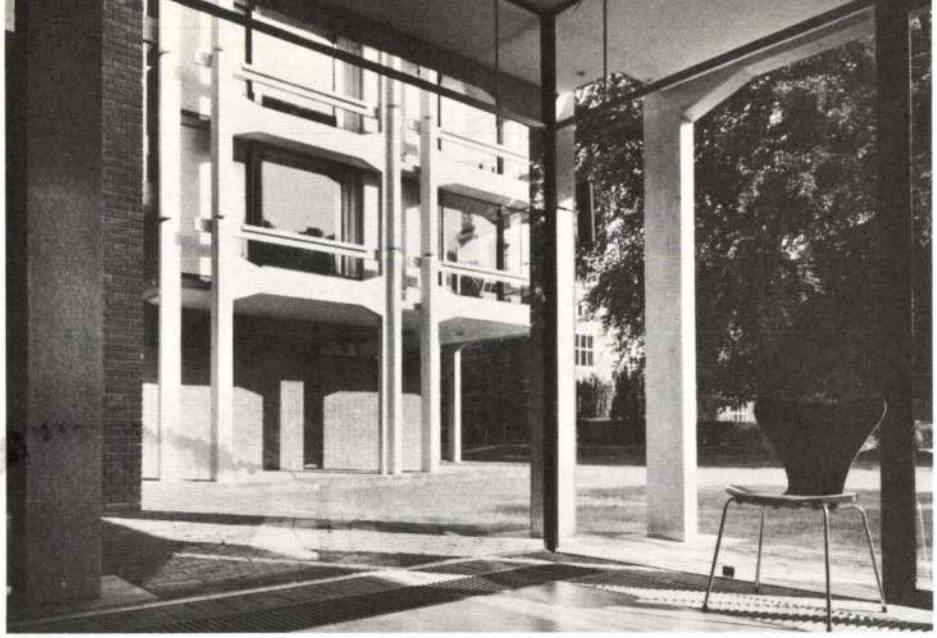
Fig. 1
Precast assembly drawing for Trinity Hall, Cambridge.

It is via the window wall that the individual student is aware that he is part of a larger community, that the building extends beyond the individual room, and that there is a quadrangle or garden surrounded by other college buildings outside. And yet this wall must also create a sense of privacy between rooms, and from outside.

The window wall

The first of these buildings was at Somerville College in Oxford, designed in 1958. The structural frame was set about 18 in. (460 mm) beyond the external wall. Precast mullions, transoms and sills held windows or precast panels. Above the transoms was a

2



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Figs. 2, 3, 4 & 5 Corpus Christi, Cambridge, Leckhampton House.

2 A view from one pavilion to the other. (Photo: Edward Leigh)

3 H frames being assembled.

4 A corner study bedroom. (Photo: Colin Westwood)

5 An example of the tooled white concrete. (Photo: Harry Sowden)

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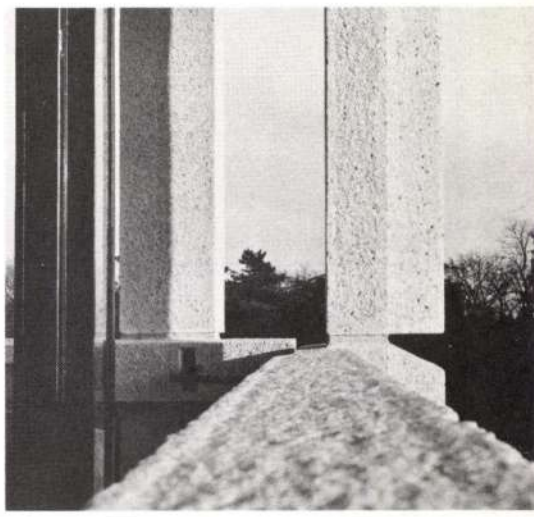

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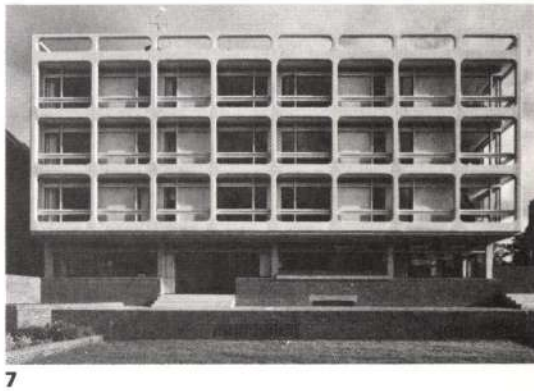

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Figs. 6 & 7 Somerville College, Oxford. (Photos: Colin Westwood)

6 Interior: Furniture arranged to form bed alcove.

7 Exterior.

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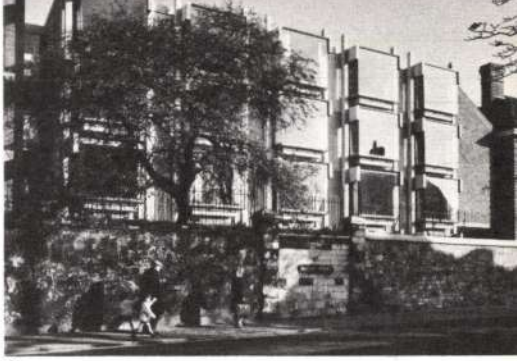

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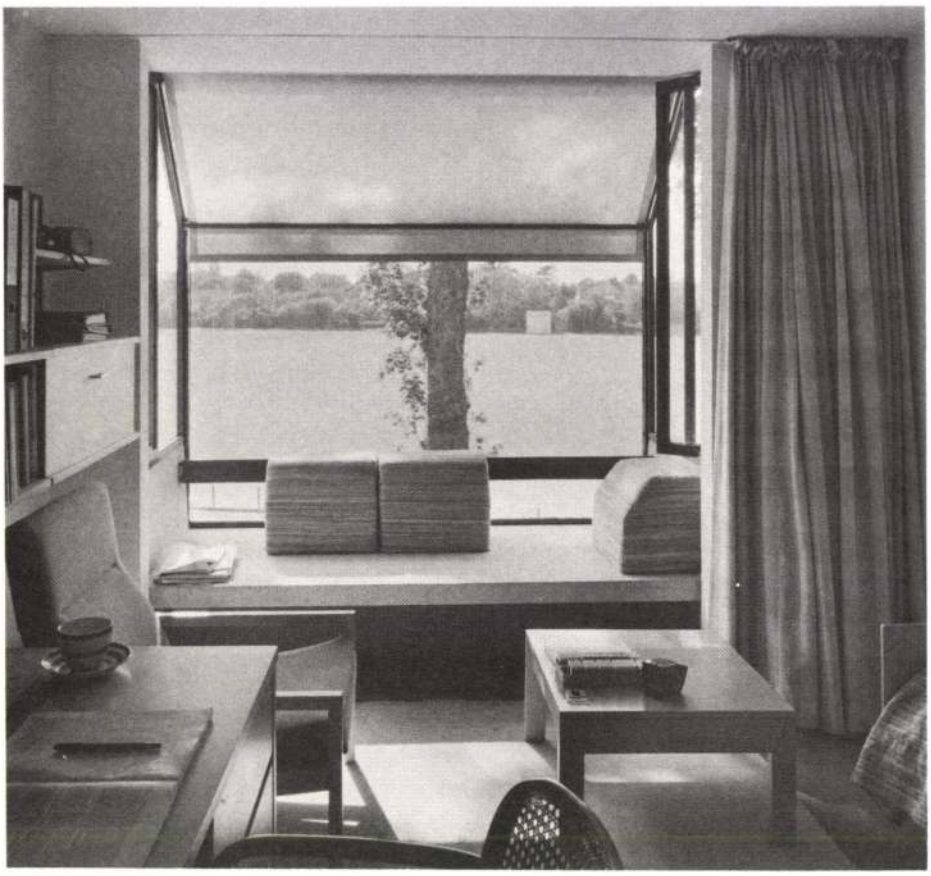
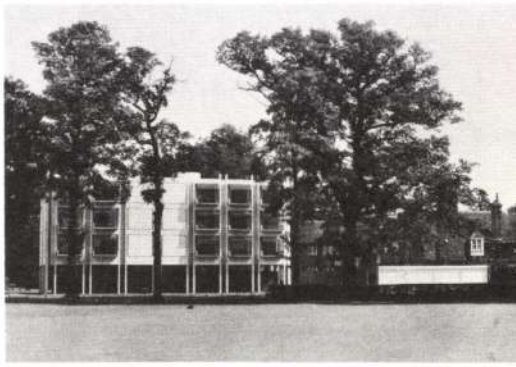
Figs. 8 & 9 Interior and exterior
The Wolfson Building, Somerville College, Oxford.
(Photos: John Donat)

10





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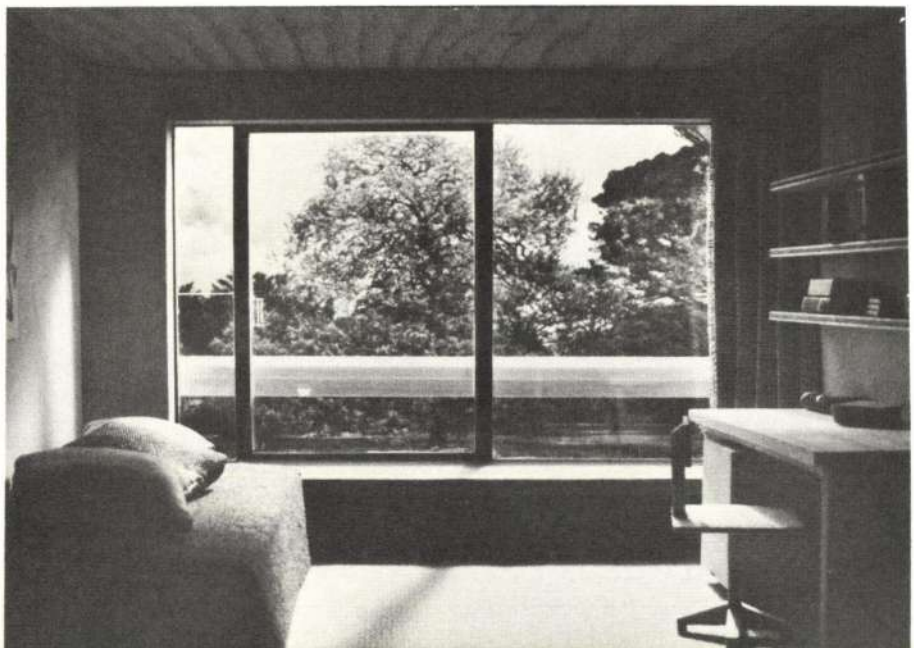
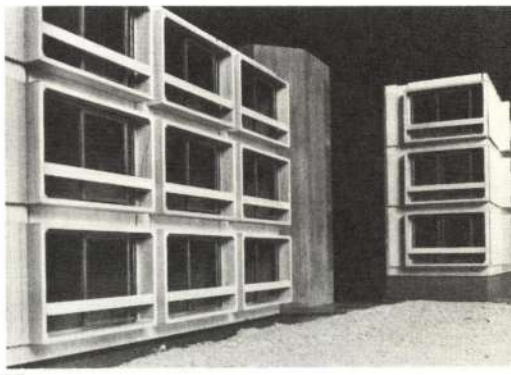
Figs. 10 & 11 Interior and exterior
Wychfield House, Trinity Hall, Cambridge.

10 (Photo: Richard Einzig)

11 (Photo: Robin Darlington)

12





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Figs. 12 & 13 Models of interior and exterior, University College, Cambridge.
(Photo: David Faggetter)

continuous clerestory, which gave the room a sense of extension by both day and night. The transom was formed to house a fluorescent light which shone upwards onto the concrete soffit, to take the glazing directly without the use of any frame, to house the sliding window track, and to carry the pelmet board and curtain track. The sill housed a heating pipe, and was grooved to take the asphalt to the gutter between the frame and the wall, and the bottom track of the sliding window. Between sill and transom there were panels, sized to take a purpose-made desk and bookcase unit, and thus allowing for alternative furniture layouts in the room. Between the panels were either large sliding windows, or smaller opening ones to give marginal ventilation. At that time the building regulations demanded a fire break between floors and the horizontal projection of the floor slabs gave this and allowed the windows to be carried down almost to the floor. This gives a generous view down into the rest of the college, and overcomes the sense of isolation which can occur in multi-storey buildings. By setting such large windows back from the structural frame, and by providing a generous handrail fixed to the frame, any sense of insecurity is avoided. For a building set in a garden, a balcony as such, seems an expensive anachronism but by sliding back the large window in the summer, the whole room is opened up.

The narrow gallery also provides a place for the window cleaner, and the overhang, by reducing sky glare, improves the quality of the daylighting and controls weathering.

At Leckhampton House for Corpus Christi College in Cambridge (1962), these ideas are largely repeated. The building is, however, planned in two linked pavilions, which give it twice as many corners. These have been glazed, so that there are views from one pavilion to the other, and through the corners to the garden beyond. This complexity of form led to a simplification of the external wall, and the mullions and transoms were omitted. (The clerestory was not liked by the lay-a-beds anyway.) On the north face, windows were omitted and the panels were pushed out to within 2 in. (50 mm) of the external frame. Within this added bay to the room, a 7 ft. (2.1 m) desk was built-in.

This idea of modulating the external wall in relation to the external frame was carried further in the next building, which was the Wolfson Building, back at Somerville College (1964). In both the previous buildings, with the external wall running parallel behind the frame, the fenestration pattern was apparently arbitrary. However the same sense of the building extending beyond the individual room could be achieved if the windows were pushed out beyond the frame, instead of pulled back behind it. Also there were obvious gains to the room in having a large window seat, which was an admired tradition in the older buildings of the university. Although all the previous buildings had had common rooms at ground level, here for the first time was the need for a column-free multi-purpose hall, and consequently a clear span. This was achieved by the introduction of precast post-tensioned concrete beams spanning across the building, with the bay windows fixed between them.

Only the sides of the bay open, so as to retain a sense of security when one sits amongst the columns and beams beyond the building. This also has the effect of turning the focus of the window seat back into the room. The introduction of beams added almost 2 ft. (610 mm) to the height of the room, which provided an opportunity to create a bed alcove with the ceiling over it dropped to the soffit of the beam. It did, however, let much more light into the rooms than we had anticipated, with a consequent loss of privacy, unless the blinds were down.

At the most recent of these buildings, that for

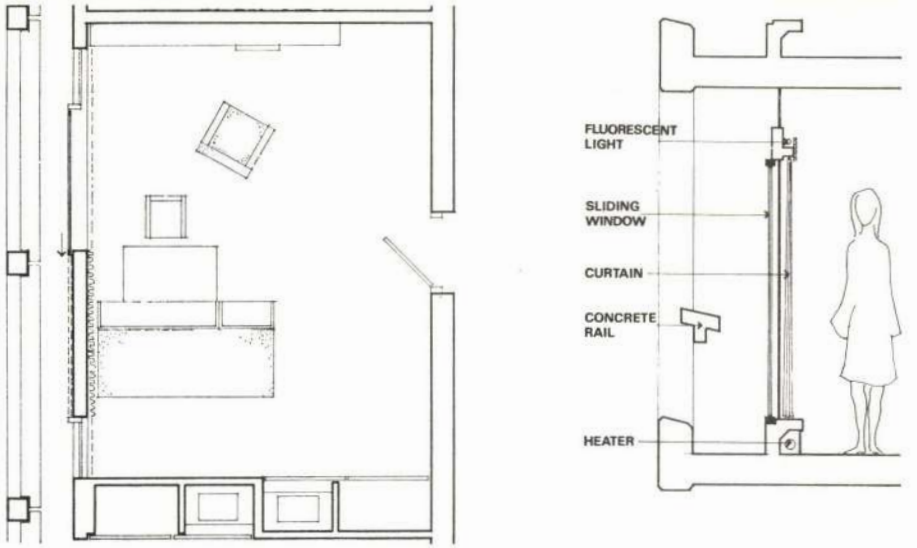

Fig. 14 Somerville College, Oxford, stages I and II.

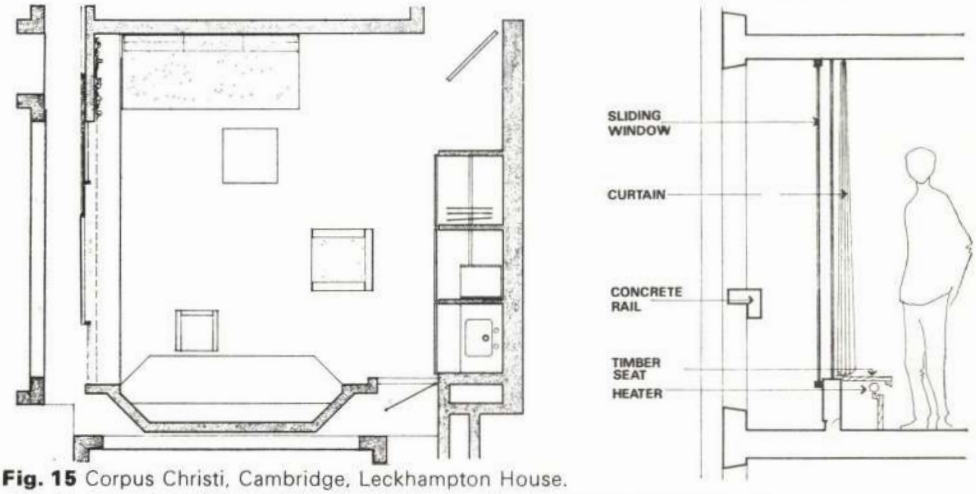

Fig. 15 Corpus Christi, Cambridge, Leckhampton House.

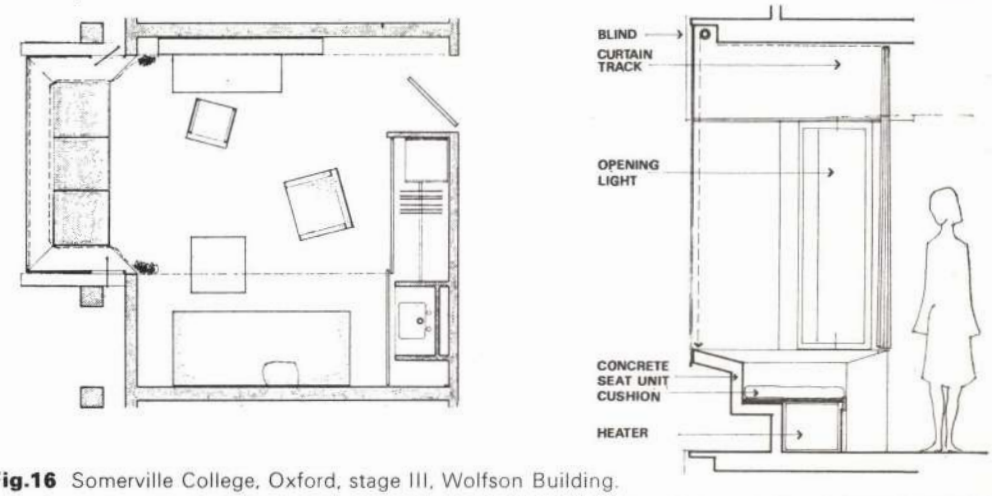

Fig.16 Somerville College, Oxford, stage III, Wolfson Building.

Fig. 17 Trinity Hall, Cambridge, Wychfield House.

Trinity Hall (1966), we have tried to simplify the details. The common room at ground level could accept internal columns, so the structure reverts to a flat slab spanning over the internal columns and out to the beams of the external frame. The tops of the bay windows were made of sloping glass, with the sun blind pulling down this slope over a relieving roller, and then down the vertical face. With the ventilation coming from the sides of the bay, it is possible to have both sun shading and ventilation at the same time. The fan convectors have been positioned under the clothes cupboard on the back wall of the room. A rail across the window acts as both a back rest and a small electric heater (250w). The latter is incorporated to cope with down draughts and condensation.

We are now working on a sixth building for University College, Oxford (1969). Here there is no common room, and so the justification for a framed structure is removed. Sound insulation between rooms is vital, so that there is much in favour of a cross-wall solution which combines the needs of structure and sound insulation. Naturally the structural change greatly reduces the number of elements of the external wall, by the omission of structural members. Two elements remain; panels and window units. The latter are designed to provide many of the solutions arrived at in the earlier buildings, privacy by their projecting sides, an enhanced quality of daylight by their projecting head and sill, outlook down into the garden combined with the security provided by an external handrail, a housing for the heating element which is back under the window and grooves to carry windows, glazing, curtain tracks, etc.

Construction of the external wall

All these buildings have been made of precast concrete. For an exposed structural frame it was a suitable material. It can be used also for cladding panels, seat units, mullions and transoms, and, by so doing, could give the buildings something of the homogeneity of the older stone buildings in the universities. It could be formed, grooved and dished so that many of the building details could be solved off the site in the precast yard.

We have made this concrete most frequently of a Derbyshire limestone (Baladon or Dowlow) with white sand and white cement. For fine elements, such as mullions (2½ in. x 5 in. (63 mm x 125 mm)), sills and window seats, it has been acid etched. For heavier structural members it has been lightly tooled. At the early Somerville building all corners were rounded to simplify this tooling, but subsequently we have preferred the slightly tattered arris which is achieved by tooling up to corners. The tooling bruises the aggregate and gives it a lighter cast.

At the earlier Somerville buildings, the panels are of a washed Derbyshire spar, but the match to the bush hammered finish of the structure is not perfect, the panels being slightly browner. At Leckhampton and Trinity Hall the panels are bush-hammered like the frame, and of the same aggregate.

Generally we like the material; it is not as harsh as calcined flint nor as soft as Portland Capstone, and cheaper than both. When treated with silicone[1] it seems to weather well, the roughened surface breaking the run off of rainwater, and the aggregate and matrix having much the same absorption characteristics.

At the Wolfson Building for Somerville the materials are slightly different. Here the frame is a smooth grey concrete, so as to relate better to the darker tones of the two adjacent brick buildings. The window seat units are made with an Aglite aggregate, which shows darker externally, and has been spun internally to give some life to the finish when seen at close quarters.

Building with precast elements has its disciplines. The early Somerville buildings were designed before it was clear where the economic advantage lay, and, therefore, without knowing if the structural frame would be precast or in situ. When it came to be built some two years later, we decided to make this frame in precast elements. To form the joints, 3 in. (75 mm) gaps were left between the column knobs and the beams, with projecting steel from both elements lapping in the gap. It was subsequently filled on site and bush hammered like the rest. The columns were cast full height on the first building, but, because of limited access for the second building, the columns were split into storey height units, and re-assembled with a simple dowel and araldite glue joint 1/32 in. (0.8 mm) thick.

In all the later buildings, the way the units are jointed is expressed. At Leckhampton and Trinity Hall, the 'H' frames rest on top of each other, and are jointed with a dowel, a metal shim, and then dry packed with mortar; the joint being expressed by a bird's mouth cut between frames. Joints between other elements have usually been formed by secret pockets and then filled in situ on site; this being a cheaper joint than the more elaborate bolted ones. However, at the Wolfson building at Somerville the beams are bolted to the columns with friction grip bolts.

The wall panels are made as thin and light as possible, since they are often the heaviest units in the building, especially the ones at the ends which enclose the larger structural spans. They have normally been made 5 in. (125 mm) thick with a ¾ in. (19 mm) polystyrene sheet of insulation cast in them. The internal face has been left fair face and decorated directly.

The external appearance of these buildings is the result of the assembly of these precast elements. The elements are made in timber moulds, which leads to flat linear elements. The elements have widely differing functions. Some are structural, some no more than window frames, and each has been sized accordingly, within the limitations of the material. Reinforcing rods have to be positioned and given adequate cover, and fixings have to be kept back from corners to avoid spalling. The building is formed by the juxtaposition of these profiled and linear elements, and its character results from the plan and proportion of these different pieces.

In the development of the design of these three dimensional elevations, study models are made to check the sizing and proportion of the elements themselves, and to refine the relationships between them. In the end these decisions are architectural rather than strictly structural. The depth of the beam at Trinity Hall has to support the window seat, and the first floor beam is 10 in. (250 mm) deeper to cover the false ceiling which was necessary at this level to lead the pipes from the rooms above, clear of the common room. The L shaped columns, and haunches at the end of the beams at Leckhampton are there to increase the vertical emphasis of the building and break the horizontal lines, which seemed appropriate for a building site amongst so many trees. How the rainwater ran off the building, especially the glass areas, needed to be controlled by splaying tops of members back to the building and forming secret gutters. The external structure led to such a gutter at each floor level, and the rainwater was either allowed to drip through from gallery to gallery until it reached the ground, or was collected at first floor level and carried to drain.

Conclusion

I have tried to show the way in which a social problem, the student's needs, led to a certain type of building, how this building was then made by the techniques which were available and how these techniques were developed both technically, as ways of jointing and forming the precast elements were improved, and also aesthetically, as the visual character of the building, which emerges from these two factors, was refined in terms of finish, proportion, and formal relationships. It is the same old recipe as Sir Henry Wotton's, who defined architecture as 'firmness, commodity, and delight.'

Credits

Job no. 1224—SOMERVILLE COLLEGE, OXFORD. STAGE I

Main contractor: Cowley Concrete Ltd.
Precast concrete suppliers: Norman Collison (Contractors) Ltd.

Job no. AA/105—SOMERVILLE COLLEGE, OXFORD. STAGE II

Main contractor: Norman Collison (Contractors) Ltd.
Precast concrete suppliers: William Sindall Ltd.

Job no. AA/114—SOMERVILLE COLLEGE, OXFORD. STAGE III

Main contractor: Norman Collison (Contractors) Ltd.
Precast concrete suppliers: St. Alban's Concrete Ltd.

Job no. 1629—LECKHAMPTON HOUSE, CAMBRIDGE UNIVERSITY

Main contractor and precast concrete suppliers: William Sindall Ltd.

Job no. AA/132—TRINITY HALL, CAMBRIDGE UNIVERSITY

Main contractor: William Sindall Ltd.
Precast concrete suppliers: Anglian Building Products Ltd.

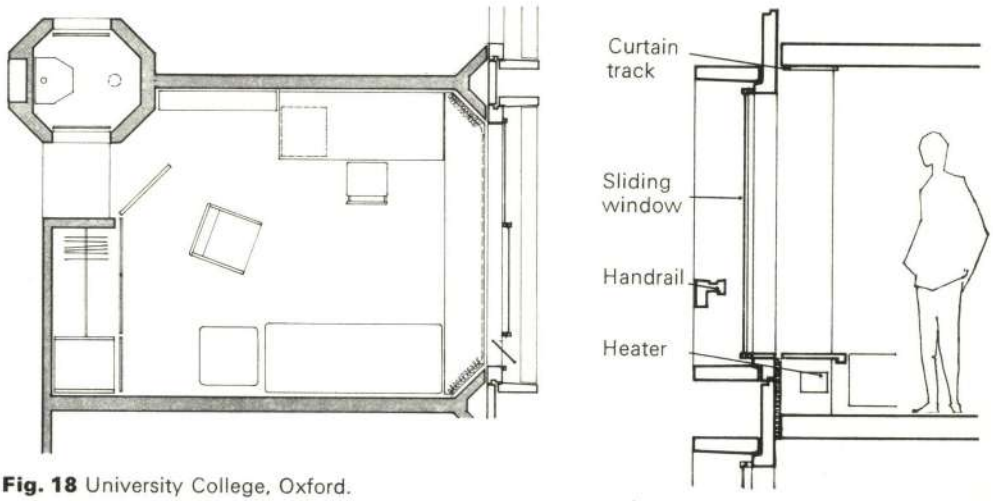

Fig. 18 University College, Oxford.

[1] 'Silicone' is a wax-like substance with silicon atoms taking the place of some or all of the carbon atoms in normal wax.