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Contents

The cover shows Penguin Books Ltd., Harmondsworth.
Photo: A. B. Gourlay

* This article which starts this issue of The Arup Journal will be discussed at the December Technical Staff Meeting. Everyone is welcome

Easing the contractors' lot*

J. M. D. Anderson

INTRODUCTION

During my time with contractors the various large sheets of paper covered in hieroglyphics (called drawings) and small sheets of paper written in some foreign language (called specifications) gave me the impression that the individuals working under the collective nom-de-plume of consulting engineeers and architects had very little idea of the problems facing the contractors' staff on site. In my two years with Arups the 'impression' has become a conviction and the 'very little idea' has become no idea at all!!

The above paragraph may seem over-critical of individuals and I am prepared to accept this but collectively it is true. Assuming you have not at this stage thrown your copy of this article on the floor and dived for the internal phone to blow your top at the author (assuming also that one of the partners has not beaten you to it and the author is no longer about), then let me explain.

When you have read all the points you will probably say 'but I know 99% of that lot'. Fair enough, so you may, but what about the other 1% ? From your point of view the article is not wasted. Consider further. The individual in front of you has probably said the same but his '1% didn't know' will in all probability be different from yours, as may be the 1% of the individual on your right and left and behind and over the other side of the gangway and upstairs and downstairs and over in number 8 or 13. There are some 700 members of our technical staff and assuming there are 6 of them whose '1% didn't know' is the same as yours and that each '1% didn't know' is shared by 7 people, then by performing the sum

$$\frac{700}{7} \times 1\% \text{ of Arups} = 100\% \text{ of Arups}$$

we find that the first paragraph is proved.

Now read on and then return and re-read this introduction.

SCOPE

The object of this article is to try to define and detail all those areas where standardizations and simplifications can ease the contractors' site problems and we, as engineers, should consider these wherever possible. Whereas the specifications for the structural works in a project are the province of the engineer, the shape is produced in varying proportions by both the engineer and architect and although we can deal with our own responsibilities in this respect we can only offer the architect advice and this should be done in cases where we think it necessary, provided always that the atmosphere is favourable.

Standardizations and simplifications should produce economies in constructional costs although these will, in the majority of cases, tend to stabilize prices rather than reduce them. In addition, on those contracts where bills are priced before the detailed drawings are available, the number and amount of claims due to out of the ordinary work will be reduced.

For the purpose of clarity I have divided the problem into six main areas, namely:

A. Excavations and foundations
B. Standard shapes and modules
C. Concrete and formwork
D. Reinforcement and stressing
E. Structural steel
F. Services and finishes

A. EXCAVATIONS AND FOUNDATIONS

1.
The average shovel is 9 in. wide so trenches less than this are impracticable.

2.
Trenching machines and excavators have given bucket

widths. It is therefore better to detail long trenches which are probably going to be machine dug to these widths.

3.
Deep narrow trenches are difficult to hand excavate, particularly by labourers with large posteriors. Consideration of any structural advantages from wider foundations should be made.

4.
In granular or similar material, boarding and strutting or battered sides will be required. In the latter case backfill will be necessary. It may be that concrete would be preferable and a lesser depth result.

5.
Small variations in the levels of the undersides of foundations of the order of 3 in. or less are difficult to achieve and usually cost more in setting out, bottoming up and checking than they save in concrete volume.

6.
Pile groups and piles within groups should be considered from the point of view of machine working positions and programming of subsequent work.

B. STANDARD SHAPES AND MODULES

7.
Queer shapes invariably cost more to make and, unless absolutely necessary, should be avoided.

8.
Shapes that are repetitive are cheaper and easier to construct since the labour gets into the swing easily.

9.
Modular construction is cheaper for the same reasons as no. 8 above.

10.
Since levelling is invariably carried out in divisions of 1/100 ft. but taped measurements where used are in divisions of 1/96 ft. it is better that where levels are given they are to dimensions common to both methods, i.e. 3 in. and .25 ft, 6 in. and .50 ft, 9 in. and .75 ft. Other levels lead to discrepancies.

11.
Dimensions as a fraction of an overall dimension which is not exactly divisible by the number of elements should be avoided. This is most common in stairs where something like 17 risers in 10 ft. 6 in. is asked for giving 7 7/17 in. per riser.

12.
Falls of drains and pipes and ditches should be kept to the respective recommendations of the British Standards concerned but at the same time should lead to round figures at manholes, interceptors, etc. It is probably better on a 38 ft. run of drain with 1 in 40 fall to give a 1 ft. difference of level at each end than specify the fall which leads to an inexact level.

13.
Where floors and slabs have to be laid to falls for surface drainage the point of no. 12 above should be noted with regard to screeds.

14.
In addition, the soffit should wherever possible be kept level since sloping formwork is comparatively more difficult to fix. Constant depth across bays can be maintained by stepping the soffit level although this is undesirable from the point of view of variable prop lengths.

C. CONCRETE AND FORMWORK

15.
The number of different concrete strengths or mixes in the work should be kept to a minimum. This is particularly necessary when ready-mixed concrete is to be used.

16.
Different concrete strengths in adjoining members which may be cast at the same time should be avoided at all times.

17.
The casting of large areas of bases, ground beams and ground slabs in one pour is undesirable since it invariably leads to prolonged exposure of the ground to the elements.

18.
The positions of construction joints should be considered at the design stage and wherever possible detailed on the drawings. This saves innumerable arguments with contractors and delays while their proposals are considered.

19.
If possible a feature should be made at construction joints. This avoids the difficulty of forming flush surfaces and reduces the chance of honeycombing of the concrete at the joint due to loss of grout.

20.
Expansion joints and watertight joints should be fully detailed on the drawings and water bars, joint fillers and sealants specified.

21.
Column sizes, wall sizes and slab sizes should, where possible, be detailed bearing in mind the standard dimensions of the materials from which formwork is made. Plywood, hardboard, etc. is normally produced in 8 ft. x 4 ft. sheets and a wall dimension of 8 ft. 2 in. x 8 ft. 2 in. will necessitate considerable wastage and also produce undesirable joint marks.

22.
In the same way as no. 21 above, timber is normally supplied in multiples of 1 in. in width rough sawn and between ⅛ in. and ¼ in. less if planed all round. Upstands, risers, etc. of 9¼ in. will produce wastage.

23.
The number of variable sizes of columns and beams should be kept to a minimum since a higher number of uses can be obtained from formwork.

24.
Formwork to sharp corners is difficult to keep grout tight and removing arrises by the insertion of a chamfered piece in the corner of the form is easier and cheaper than grout-proof construction. In addition the likelihood of subsequent damage is minimized.

D. REINFORCEMENT AND STRESSING

25.
The number of different bar types and diameters should be kept to a minimum.

26.
Reinforcing rods are normally produced in 30 ft. and 40 ft. lengths. Where possible bars should be kept to fractions of these. This minimizes wastage. It is more expensive usually to detail a bar 29 ft. 6 in. long as 30 ft. than cut off 6 in. The 6 in. will be charged anyway as scrap.

27.
Laps should, where possible, be designed to anticipated lift heights or slab sizes.

28.
Bar centres should be simple measurements. 6 in. is better than 5⅞ in.

29.
The relative positions of longitudinal and vertical steel in mats in narrow sections should be considered from the point of view of vibrators. In a 6 in. web of a prestressed beam with two-way mats on each face of ⅜ in. bars the longitudinal bars are better on the inside to give space between the mats and formwork for vibrator insertion.

30.
Small diameter bars, ¼ in. and ⅜ in., are always detested by steelfixers. Bar fixing costs are more relative to length than weight. Therefore larger diameters at larger spacings are to be preferred to smaller diameters at smaller spacings.

31.
Cover to reinforcement should be constant to minimize the manufacture of spacing blocks.

32.
Support chairs in slab mats should be detailed where possible.

33.
Congestion in small sections is always a problem and should be avoided. Sometimes vibrator insertion proves impossible.

34.
Bars protruding through stop end formwork or starters for subsequent structures are extremely difficult to accommodate particularly when striking formwork. These bars should be kept to a minimum or detailed to be bent up at the formwork face.

35.
In prestressed members the contractor will invariably want to suspend cable ducts from the reinforcement. Due allowance should be made for this eventuality.

36.
In prestressed members access for threading strand and using jacks must be considered during detailing.

37.
Isometric views of congested and complicated corners would help steelfixers (and also detailers).

E. STRUCTURAL STEEL

38.
One of the biggest problems with structural steelwork is the large variety of bolt diameters and lengths. Careful bolt group design and the use of a minimum number of bolt lengths with added washers makes stores and erection problems simpler.

39.
All bolts have to be tightened. Room must be given particularly for high strength friction grip bolts.

40.
The number of structural sizes should be kept to a minimum.

41.
The position of holding down bolts must be checked against the reinforcement drawings. More often than not holding down bolts foul main rods.

42.
Holding down bolts should be detailed to give horizontal play. They seldom end up in the exact position and the 1/16 in. oversize on holes is not enough to take care of this.

F. SERVICES AND FINISHES

43.
Fixings for finishes and holes, recesses etc. for services should be detailed wherever possible on structural drawings. This is not always possible but a maximum of information should be obtained.

44.
The positions of reinforcing rods should be such that the fixings and holes etc. are clear.

45.
Topping slabs should be thick enough to take electrical conduit, water pipes, heating pipes etc. one on top of the other at intersections. It is not uncommon to find $1\frac{1}{2}$ in. topping containing a $\frac{3}{4}$ in. water pipe and $\frac{3}{4}$ in. conduit crossing at right angles.

46.
Pipes through water-tight walls must have puddle flanges.

CONCLUSION

A great deal of the foregoing will apply on your contract. Have you considered it all? In addition, do your drawings cross check? Have you included *all* possible information? Are they clear? Have you considered the contractor's possible method? His crane position? His access? His huttage area? The standard of his workers? Finally, have you done all you can to ease his lot?

Tower Hill development

Tom Henry

INTRODUCTION AND GENERAL DESCRIPTION

The site of this development lies on the west side of Tower Hill opposite the Tower of London.

Because of its prominent position the scheme had to be approved by the Fine Art Commission. To this end a proposed tower block lost six floors to become a mere 14 storeys high, off the podium slab. Also on the podium slab stands an S-shaped office block, five storeys high, with shops at podium level under the S-block.

The height of the tower block is 163 ft. and the S-block 55 ft. above podium level. The tower block is 105 ft. x 82 ft. and the S-block 740 ft. by a width of 51 ft. 6 in. Two open courtyards are formed by the shape of the S-block (Fig. 1). The podium area is approximately 360 ft x 220 ft.

Below the podium are store rooms for the shops above, together with a three-level underground car park. The lowest car park level is for the use of the office personnel while the upper two parking levels are for car parking and single decker buses carrying tourists to the Tower of London.

The buildings are of reinforced concrete construction with precast exposed aggregate mullions and cladding panels.

DESIGN AND DETAIL

It is not the intention to dwell on design or details of a general nature that comprise most run of the mill work but to describe particular points of interest in the unusual.

TEMPORARY WORKS

The site is bounded by Great Tower Street to the north and Lower Thames Street to the south and falls 20 ft. in

Tower Hill Development, view looking north-west (Photo: Edwin Lewzey & Co.)

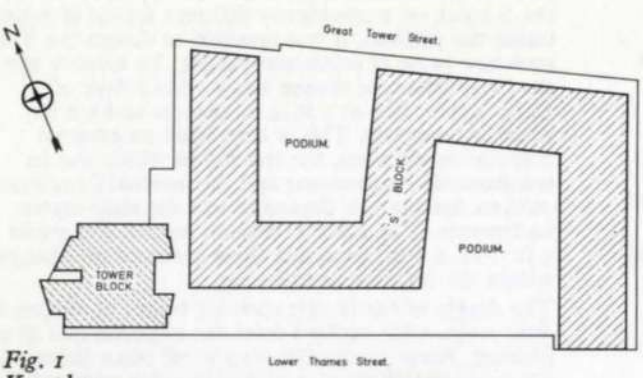


Fig. 1
Key plan

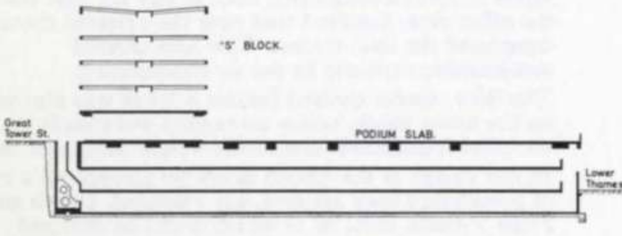


Fig 2
Cross section, north-south

formation level, creating a 30 ft. deep basement against Great Tower Street and 10 ft. depth at Lower Thames Street (Fig. 2).

On appointment of Geo. Wimpey & Co. to the contract, discussions took place regarding the retention of Great Tower Street and the 30 ft. excavation. The following system was adopted. 10 in. x 10 in. x 49 in. universal beam sections, termed H piles, were driven at 12 ft. 6 in. centres around the north, east and west boundaries of the site to a depth of approximately 15 ft. below basement level. The central 'dumpling' was then excavated, leaving the sides raked to a natural fall.

The top 5 ft. around the perimeter were then removed and 3 in. thick timber sheeters were applied horizontally to the H piles and retained by Wimpey patent clips to the H piles (Fig. 3).

As hand excavation progressed down the face of the H piles, so successive sheeters were applied (Fig 4). The top of the H piles were retained in position by welding horizontal waling beams of 12 in. x 12 in. x 92 in. universal beam sections to the H piles with raking shores of *Larssen* BP 4 B. Box piles are 25 ft. centres raking down to mass concrete buttresses, which are cast below basement level. A flat jack was used between raking shore base plates and the concrete buttresses to stress the raking shores before excavation progressed too far on the retaining wall face.

WATERPROOFING OF BASEMENT

This is generally obtained by use of an external membrane in the form of asphalting or internal membrane in the form of a waterproof cement rendering.

In this case with difference in levels of 30 ft. below ground formation level at the north side and only 10 ft. below ground formation level at the south side, the general flow of water found present at the north-west corner was from north to south towards the river.

We proposed using a sub-basement slab of 10 in. thick no-fines concrete with a herring-bone pattern of 12 in. wide x 10 in. deep ducts carrying the water under the 5 in. reinforced concrete basement slab through the no-fines slab to an agricultural drain running the free length of the south side of the site (Fig. 5).

To prevent cement grout from the 5 in. reinforced concrete

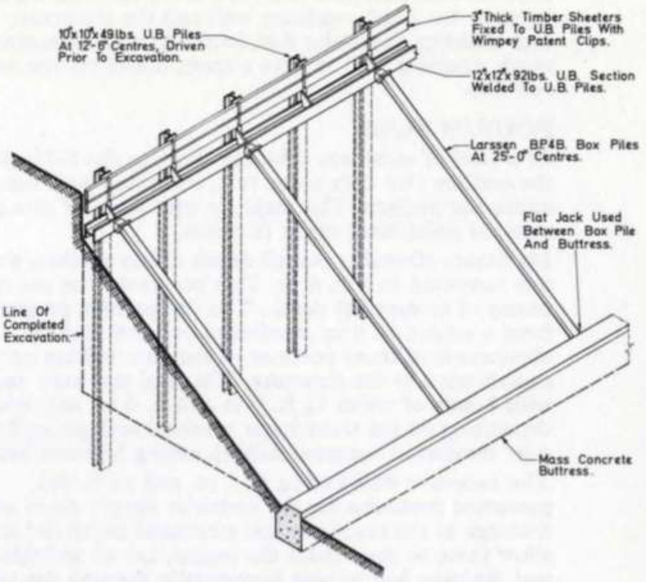


Fig. 3
Extent of excavation at erection at Larssen box piles

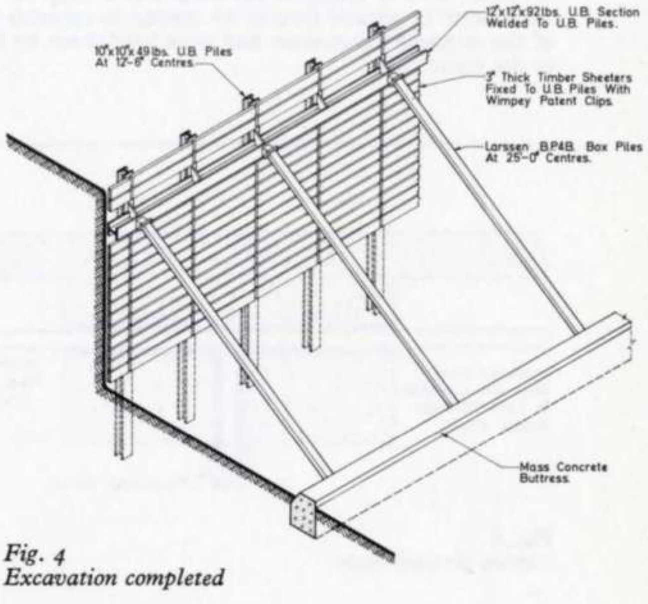


Fig. 4
Excavation completed

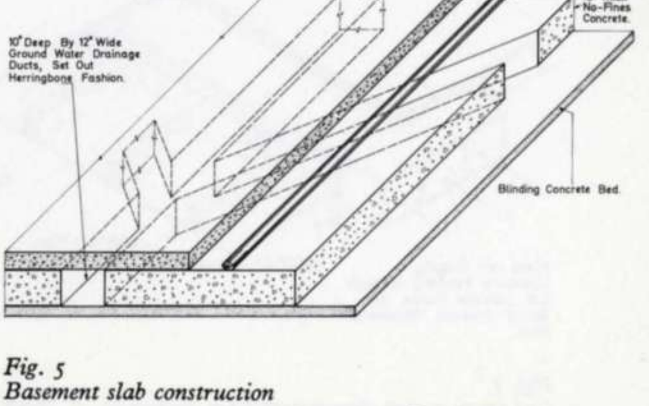


Fig. 5
Basement slab construction

basement slab filling the voids in the no-fines slab underneath, a separating layer of visqueen sheeting was laid. No-fines concrete was used to fill the working space between the north retaining wall and the temporary sheet wall enabling any water that found its way up against the north retaining wall to have a ready outlet via the no-fines sub-slab.

PODIUM SLAB

Of a total of some 200 columns carrying the S-block on the podium slab only some 10% coincided with columns under the podium. The majority with loads of 200-400 kips acted as point loads on 25 ft. spans.

Maximum allowable overall depth of any podium structure was restricted to 3 ft. 6 in. This prevented the use of beams of economical depth. The design then progressed from a solid 3 ft. 6 in. reinforced concrete slab to elimination of those portions of the slab making no useful contribution to the structure. The final structure ended with beams of width 14 ft. 6 in., 12 ft. 6 in. and less, depending on the shear loads involved and generally 9 in. reinforced concrete slabs spanning between beams.

The excessive width of 14 ft. 6 in. and 12 ft. 6in. presented problems for the sprinkler supply pipes and drainage as the restriction on structural depth did not allow these to pass under the beams, i.e. all sprinkler pipes and drainage had to pass horizontally through the podium beams. A further complication arose in that maximum spacing of sprinkler heads at 8 ft. 6 in. meant that the beam soffits had to have sprinklers.

Provision was made to pass the 6 in. and 4 in. diameter sprinkler supply pipes and the 9 in. and 6 in. cast iron drainpipes horizontally through the beams and at beam centre lines vertical duct drops were made to tee off a sprinkler head to sprinkler the beam soffits (Fig. 6). These ducts were in general formed by casting in asbestos pipes of the required dimensions and were held down by fixing to the shear links.

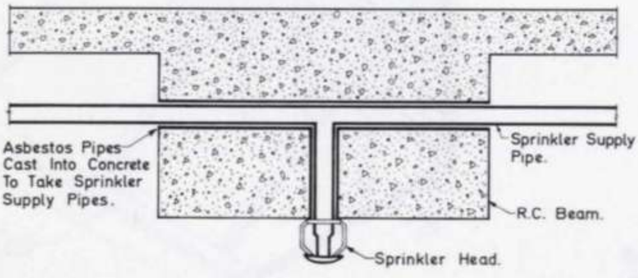


Fig. 6
Section through beam

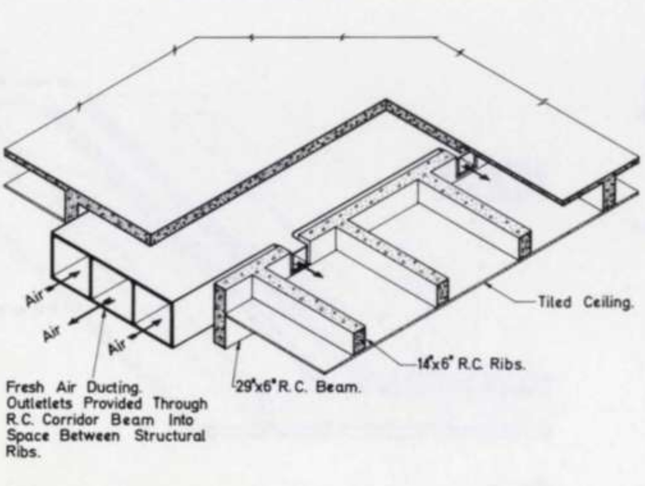


Fig. 7
Cut away section showing corridor ducting in 'S' block

OFFICE BLOCK SUPERSTRUCTURE—S BLOCK

Because of the restriction in structural depth of the podium structure and the need to carry the structure of the S block on a completely different layout of columns under the podium, it was essential to design the S block structure to be of minimum weight. To achieve our aim, the basic structure chosen was a ribbed floor of 14 in. x 6 in. ribs at 3 ft. 4 in. centres and a 2 in. structural topping. This is supported on external columns or mullions, the spacing of which was an architectural requirement and the internal (corridor) column spacing was chosen to suit the shop layout underneath. The internal column section was made 4 ft. 1 in. x 6 in. to enable these columns to 'disappear' within the corridor partitioning.

The design of the in situ corridor beams of section 29 in. deep x 6in. wide evolved from the requirements of corridor ducting. Fresh air conditioning to all office floors required a 2 ft. minimum depth and full corridor width of 5 ft. 6 in. for the flow and return ducts (Fig. 7). At 3 ft. 4 in. centres holes 9 in. x 9 in. were formed in the in situ corridor beams to which metal trunking could be sealed at the required spacing of the corridor side. The tiled ceiling in the office floors was then sealed up to the rib soffits to form a readymade duct in any rib void throughout the office area. Louvred tiles near the external elevation completed the duct system. This idea created considerable economy in the air conditioning.

The floor system evolved for the S block was also used on the tower block, where economies were made on the piled foundations due to the weight saving on the floors.

In our design of the ribbed floors we considered a number of proprietary floor systems, e.g. *Pierhead*, *Omnia* and *Fram T*-beam units all of which could be obtained at a similar price. Taylor Woodrow chose, however, to obtain the precast ribs from Stent Precast Concrete Ltd.

EXTERNAL COLUMNS

External columns to both blocks were also precast and were supplied by Stent Precast Concrete Ltd. The largest of these on the tower block carrying 550 kips were 21 in. x 18 in. shaped and had three pockets 5 in. x $2\frac{1}{2}$ in. cross section cast in the base of the column to take six splice bars in pairs of 1 in., $1\frac{1}{4}$ in. or $1\frac{1}{2}$ in. diameter. The largest precast columns weighed just over $1\frac{1}{2}$ tons.

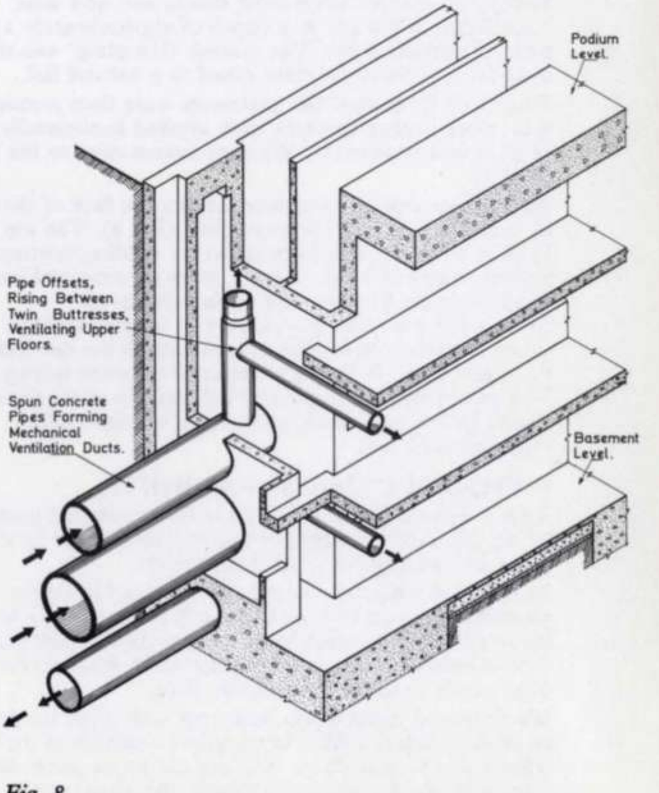


Fig. 8
Distribution of mechanical ventilation ducts

MECHANICAL SERVICES

If the three-level underground garage were designed on the basis of the existing Fire Regulations of the L.C.C. Building Byelaws at that time and due regard paid to the required free air ventilation, there would have been practically no podium slab left. A compromise was reached in that as much free air ventilation as possible was to be provided.

Mechanical ventilation was also quite extensive and ducts with cross sections of 15-20 sq. ft. were required. These sizes, however, could not be accommodated under the floor slabs due to the restricted headroom of the coach and car park.

This problem was overcome by:

1.
The placing of these ducts in the form of large 5 ft., 4 ft. and 3 ft. diameter spun concrete pipes under the basement slab and rising as small ducts up the column face.

2.
By feeding similar large diameter spun concrete pipes as ducts along the north retaining wall at basement level with vertical risers to the upper two floors between the pairs of buttresses that occurred every 25 ft. and distributing across the soffit of the floor slabs in smaller sized ducts (Fig. 8).

IN RETROSPECT

Taylor Woodrow must have liked the structural scheme for the office blocks as they suggested this form of design to our Edinburgh office for one of their development schemes being built by Taylor Woodrow.

The precast mullion system was again suggested by Taylor Woodrow in another multi-office block development in Victoria Street for which we were not the engineers.

Tower Hill Development, view looking north-west (Photo: Edwin Lewzey & Co.)

Fire damage to a reinforced concrete structure

Ben Glover

'One, two, plenty'—Tasmanian method of counting

SYNOPSIS

A fire in a section of Block 7 at Cwmbran Town Centre on 21 March 1967 presented the problems of assessing the extent of structural damage and the most economical way of making it good.

Literature on assessing and making good fire damage to a reinforced concrete structure is lacking in specific detail. The effect of heat on the strength of concrete and steel has received a certain amount of attention. However, the available information was insufficient to enable one, in this instance, to determine, after only a visual inspection, what should or should not be replaced in what appeared to be border-line cases.

It was therefore decided to test the damaged materials individually and to load test a part of the structure.

From the results of these tests the repair work comprised the anchoring back of the ends of the exposed main reinforcing steel and the rebuilding of the concrete, by means of guniting, to its original line.

THE FIRE AT BLOCK 7

The structure

The part of Block 7 at Cwmbran which was affected by the fire is of reinforced concrete construction, comprising 22 in. x 22 in. columns supporting a 12 in. solid and waffle first floor slab, with a 17 ft. hollow pot cantilever canopy on the outer edge. Solid slab bays have a central 10 ft. x 10 ft. stair opening. Where these openings are not used they are blanked off with prestressed concrete planks. The bay in which the fire occurred was the only one where timber planks had been used. The columns have a $\frac{3}{8}$ in. *Carlite* lightweight plaster finish.

The area is bounded on three sides by 9 in. solid brick walls and the front face, under the canopy, by a shop plate-glass window.

The concrete design strength was 4,500 psi at 28 days. The coarse aggregate used was a mixture of gritstone, sandstone, chert, flint and quartz with traces of shell, together with a dredged sand. The reinforcing steel was high tensile *Square Grip*. The relevant part of the structure was 16 months old at the time of the fire.

The fire

The fire occurred in a ground floor linen shop at night, burning for some hours before being brought under control.

The shop was completely gutted, causing extensive damage to the soffit of the first floor slab. The reinforcing steel in the underside of the slab had been exposed over extensive areas due to the concrete exploding off, Fig. 4 and Fig. 6. Evidence of calcinating of concrete was found.

None of the columns was structurally affected, the plaster having protected them satisfactorily.

Although the timber planks in the stair opening were burnt out, causing a flue, the fire did not spread to the upper floor.

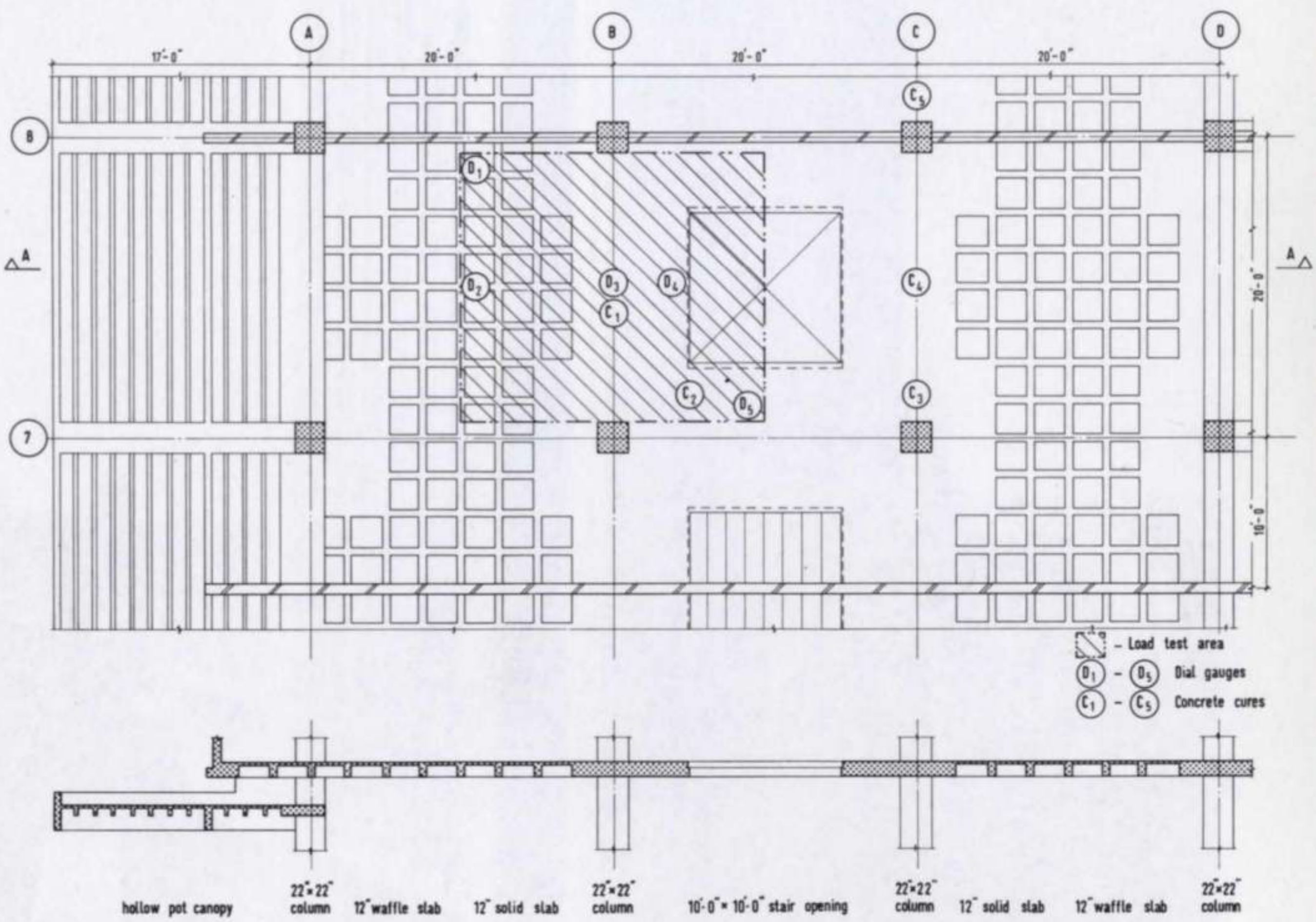


Fig. 1
Soffit first floor
(Illustrator: Marion Raine)

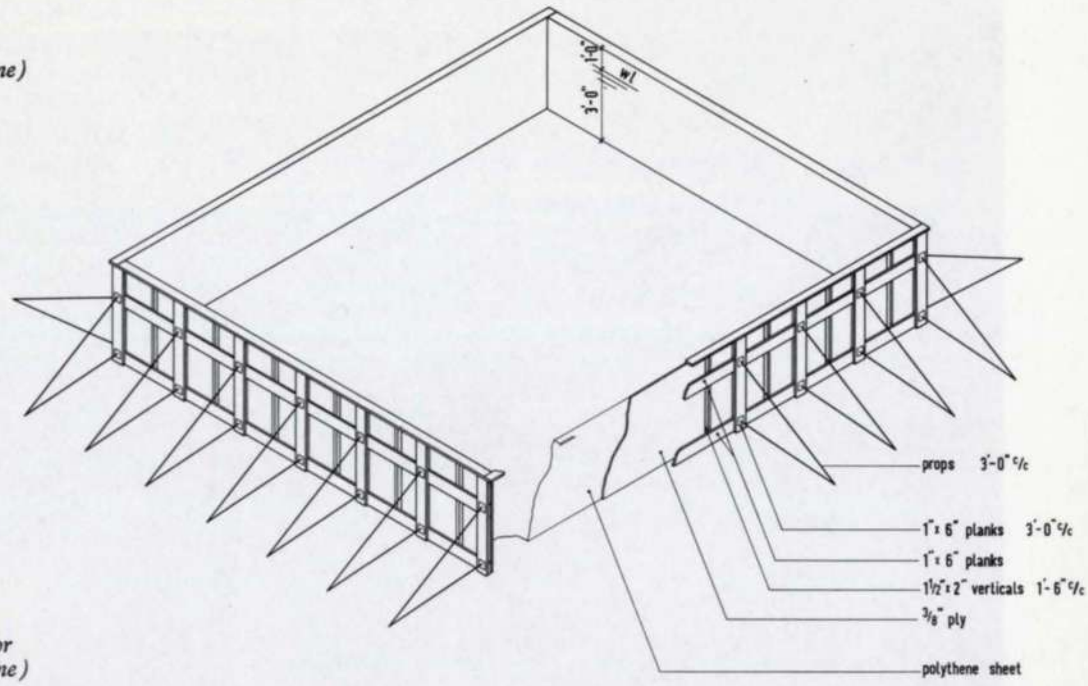


Fig. 2 right
Load test tank
(Illustrator: Marion Raine)

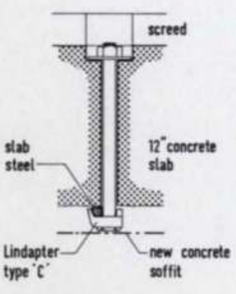


Fig. 3 above
Detail of Lindapter anchor
(Illustrator: Marion Raine)

The brick walls, which do not contribute to the strength of the structure, were badly spalled and cracked. However, they did successfully retain the fire within their bounds.

The maximum temperature was estimated to be in the region of 800°C (1475°F).

From visual inspection it appeared that the slab strength may have been reduced below its structural design strength. If that proved to be the case there were three methods of repair to the concrete structure available:

(a) Cut out and recast the slab
(b) Incorporate new supports in the form of fireproofed steel or concrete members to take the live load
(c) Remove the damaged concrete, anchor back the main reinforcement and rebuild the concrete to its original line to enable the slab to work as designed.

Investigation of fire damage

To enable a final assessment of the structural damage, it was decided to test the damaged materials individually and, depending on the results, it would be determined whether a load test should be carried out.

Firstly as a safety precaution, live loads were kept off the affected area and no new dead load added. The slab was propped with 4 ton props at 6 ft. centres.

Tests

Five 6 in. diameter concrete cores were cut from the slab No. 1-4 from the damaged area and No. 5 from outside the area, to act as a reference—positions shown on Fig. 1. Three sections of exposed reinforcing bars were also cut.

These samples were tested, the results are given in Table I and Table II respectively.

TABLE I. CONCRETE DENSITY AND STRENGTH

Core No.	1	2	3	4	5
Bulk density lb./cu. ft.	152.6	153.4	151.9	153.4	149.6
Cube strength psi at 16 months	9,300	7,450	9,850	10,800	6,300

TABLE II. REINFORCING STEEL

Bar No.	1	2	3
Diameter	⅝in.	⅜in.	½in.
Ultimate tensile strength psi	78,200	77,500	82,000
Yield point psi	65,450	70,500	70,800
Elongation %	21.8	15.3	14.0

As the samples tested indicated that the materials were still satisfactory, it was decided to load test an 18 ft. × 20 ft. section of the worst affected area, see Fig. 1.

The slab was designed for a live load of 120 lb./sq. ft. As all the dead load was already applied, it was decided that the load test should be 1.5 the live load, i.e. 180 lb./sq. ft. The criterion of the slab's structural value was based on a comparison between the calculated deflexion and the actual deflexion under load and the recovery on removal of the load.

Using Timoshenko's '*Theory of Plates and Shells*' the central deflexion for homogeneous slab, gauge 3 Fig. 1, was calculated to be 0.049 in. for the load test.

A timber tank, lined with a polythene sheet, Fig. 2, was constructed over the load area. Deflexion gauges were set out on the soffit of the slab in positions shown in Fig. 1 and the props released to give ¼ in. clearance.

The tank was filled with water at a load rating of 120 lb. sq. ft./hour.

The recorded deflexion at gauge 3 when the slab had been subjected to the load test for twenty-four hours was 0.059 in. There was an immediate recovery of 74% when the load was removed. Other deflexions and their recoveries are given in Table III.

TABLE III. LOAD TEST

Dial Gauge	1	2	3	4	5
Maximum deflexion in inches	.028	.055	.059	.021	.016
% Recovery	43	73	74	90	98

The low recovery of position gauge 1 was attributed to the breeze-block wall at first floor level which prevented the slab from recovering.

From the results of all the tests it was deemed that the structure, if made good, was structurally acceptable.

Repair procedure

The slab remained propped during the repair work.

A specification for the remedial work was drawn up and included—the soffit to be hacked to remove all spalled, loose, fractured or otherwise fire damaged concrete and cleaned to a sound concrete face; the steel to be cleaned of rust and scale by means of grit blasting; the tying back and anchoring the ends of the main reinforcing steel by means of *Lindapters* (see Fig. 3) thus enabling it to develop working stresses, and to rebuild the concrete by the application of guniting to its original line, ensuring full bond

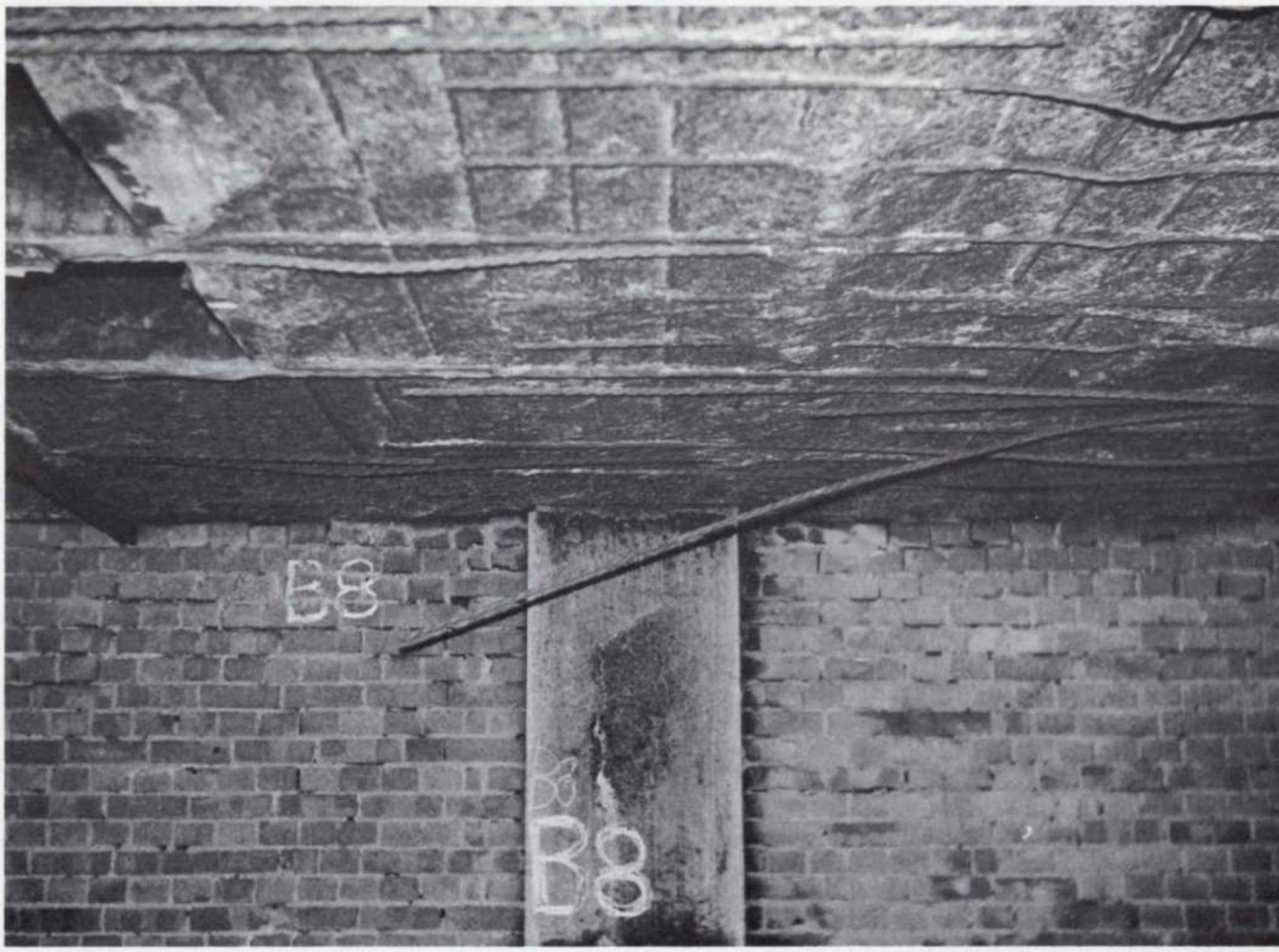


Fig. 4
Damaged soffit solid slab

between old and new concrete and new concrete and reinforcing steel. Where the thickness of the applied gunite exceeded 1 in., additional mesh reinforcement of 0.0025 times the cross sectional area of gunite was to be inserted.

The finish was left as applied (Fig. 5 and Fig. 7) and no attempt was made to float off as it could cause structural damage such as introducing cracks and breaking of bond.

Costs

The total plan area of slab which was structurally repaired was 200 sq. yds. The cost of propping, cleaning, additional reinforcement and guniting was £2,543. The cost of the load test was £320. The overall rate for the structural testing and repair of the slab being £14.6.6. per sq. yd.

THE EFFECTS OF FIRE ON A REINFORCED CONCRETE STRUCTURE

Often when a reinforced concrete structure has been subjected to a fire an urgent assessment of the damage and its structural significance is required after a visual inspection only. Also, to be able to properly plan a detailed investigation it is useful to have some background knowledge of the likely effects of fire on both concrete and steel.

Concrete

Concrete which has been subjected to temperatures up to 300°C (575°F) should not be adversely affected. At temperatures greater than 300°C (575°F) concrete usually loses strength due to the changes and deterioration that the water, aggregates and cement undergo.

Water

Damage can be caused by water in the pores becoming superheated above the boiling point of water with violent results. The more the concrete has dried out the less it is liable to suffer this type of damage.

Fig. 5
Repaired soffit solid slab
(Photographer: Ben Glover)

Aggregates

Generally because of the easier readjustment of the fine aggregates, the effect on it when subjected to fire is limited. The coarse aggregate is much more adversely affected and usually the main cause of serious concrete spalling.

Chert and flint when heated to over 250°C (480°F) show a colour change similar to that of sandstone. At slightly higher temperatures, shattering due to expansion occurs.

Quartz particles begin to expand rapidly at 580°C (1070°F).

Igneous rocks generally do not show a colour change on heating. Granites sometimes crack or shatter at temperatures above 580°C (1070°F) through quartz expansion. Basic types (dolerite, basalt) show no effect at this temperature, but may show expansion effects when heated above 900°C (1650F°).

Limestone is one of the better natural aggregates from the point of view of fire resistance.

Many limestones undergo a colour change at about 250°—300°C (480°—575°F) to a marked pink or red. The colour sometimes increases in depth at higher temperatures up to 600°C (1115°F). At temperatures above 700°C (1290°F) calcination of the limestone occurs, becoming rapid at 900°C (1650°F). When calcination occurs the red colour disappears and the rock slowly disintegrates.

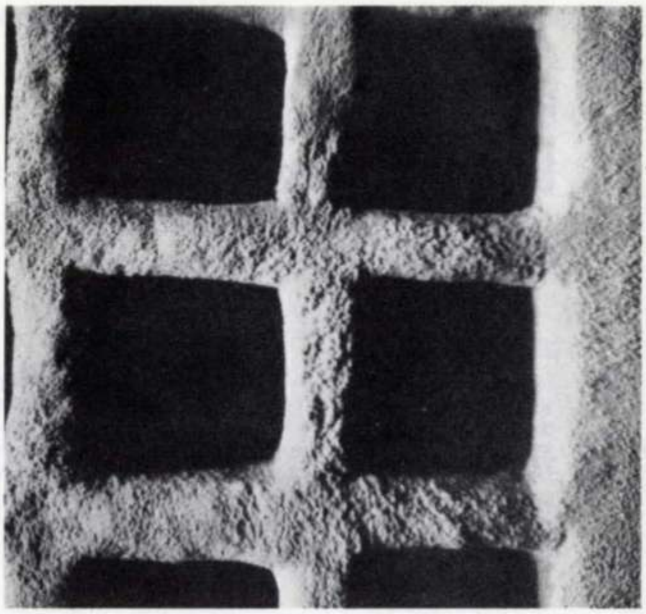


Fig. 7
Repaired soffit waffle slab
(Photographer: Ben Glover)

Fig. 6
Damaged soffit waffle slab

Cement

When hardened concrete is subjected to temperatures of 900°C (1650°F) and over, re-calcination of the cement occurs and a thin white powdery surface is discernible on exposed faces.

Cracks occur roughly parallel to the face of the concrete and layer after layer of concrete spalls off. The depth varies approximately with the length of exposure and increases about 1 in. for every three hours of fire duration.

Reinforcing steel

It is unlikely that reinforcing steel still encased in concrete will be affected. However, if the concrete has spalled off it can be expected that both the elastic limit and the ultimate strength have been reduced if the temperature exceeded 400°C (755°F).

The behaviour of mild steel when subjected to high temperatures varies according to its original properties, especially the carbon content. Generally for temperatures up to 400°C (755°F) there is an increase in tensile strength with a corresponding loss in ductility, the percentage of elongation being reduced. At temperatures greater than 400°C (755°F) there is a marked dropping off of tensile strength until at 600°C (1115°F) only about 20% of the original strength remains. The yield strength similarly reduces with increases in temperature above 400°C (755°F).

The effect of high temperatures on high tensile steel is also to reduce its tensile value. However, the temperature at which this occurs depends on the method of manufacture and mechanical properties of the bar.

Cold worked bars are liable to become annealed at temperatures above 400°C (755°F), after which they will exhibit properties similar to mild steel but with a reduced elongation.

Hot rolled bars should not be too adversely affected until their original annealing temperature is reached. This is usually in the region of 550°C (1250°F), after which there is a reduction in their tensile strength.

Reinforcing steel which has been subjected to temperatures of 400°C (755°F) plus, should be treated as suspect and tests carried out on the affected bars.

CONCLUSIONS

The first reaction on surveying a smoke-blackened structure after a fire of any magnitude, is often one of incredulity that it is still standing, which then gives way to deep despair that nothing can be salvaged.

However, a cool-headed investigation will usually show that a lot of the structure can be saved and the damage repaired, due thanks being accorded to such things as '*Fire Regulations*'.

Warehouse for Penguin Books Ltd.

A. B. Gourlay

THE BRIEF

In April of 1966 Arup Associates were asked to design a warehouse of some 90,000 square feet for the storage of paperback books. The warehouse was to be sited on an open field beside the existing buildings in Harmondsworth. In addition to the warehouse, car parking facilities, lorry loading and unloading bays were required. Our clients were particularly keen that the entire area should be well landscaped.

THE BOOKS

The paperbacks, which arrive by road in 4 ft. cube pallets from printing works in the Midlands and the South, are stored in a bulk warehouse. As orders are received, books are taken by truck from the bulk store to a packaging department where they are made up and subsequently despatched to book stores all over the world.

Our clients, in conjuction with Personnel Administration Ltd., carried out a considerable amount of research into the trucking problem. A new swivel-headed fork lift truck enabled pallets to be placed at right angles to the line of travel. Operation could now be within a 5 ft. 6 in. aisle width instead of the 8 ft. aisle with a 4 ft. deep stock area either side. This gave a total lateral module of 13 ft. 6 in.

The pallets would be stacked five high, the bottom three pallets being onto racking so that they could be broken up for hand selection and the top two pallets being bulk storage. This gave an overall height of 27 ft.

The entire warehouse floor would have to be level with the existing warehouse, which was to be converted to a modernised packaging department.

The warehouse was to be heated, as damage would occur to the glue used in the bindings, if the temperature were to fall to freezing. The clients asked for a high level of natural daylight. This proved to be a decisive factor in the design of the building as a bulk store of this type could be designed without any form of natural lighting.

PLANNING

Having started the design in April, we were well under way in July when the Government credit squeeze forced our clients to reduce the scheme by one third to 60,000 sq. ft.

The new warehouse was sited away from the existing buildings and a link block connecting the buildings provides a covered passageway as well as a lorry-loading bay.

The space between the buildings forms a generous lorry turning area. A one-way road takes traffic around the building and serves both the new and existing premises.

By setting in the south wall at low level, a covered way is provided from the new car park to the existing premises.

The surrounding five acres of land are to be landscaped. The top soil cleared for the foundations of the building forms a mound in the south-west corner of the site and there is a lake in the north-east corner. The external area is to be grassed, trees and shrubs are to be planted and arranged so as to re-create a natural setting.

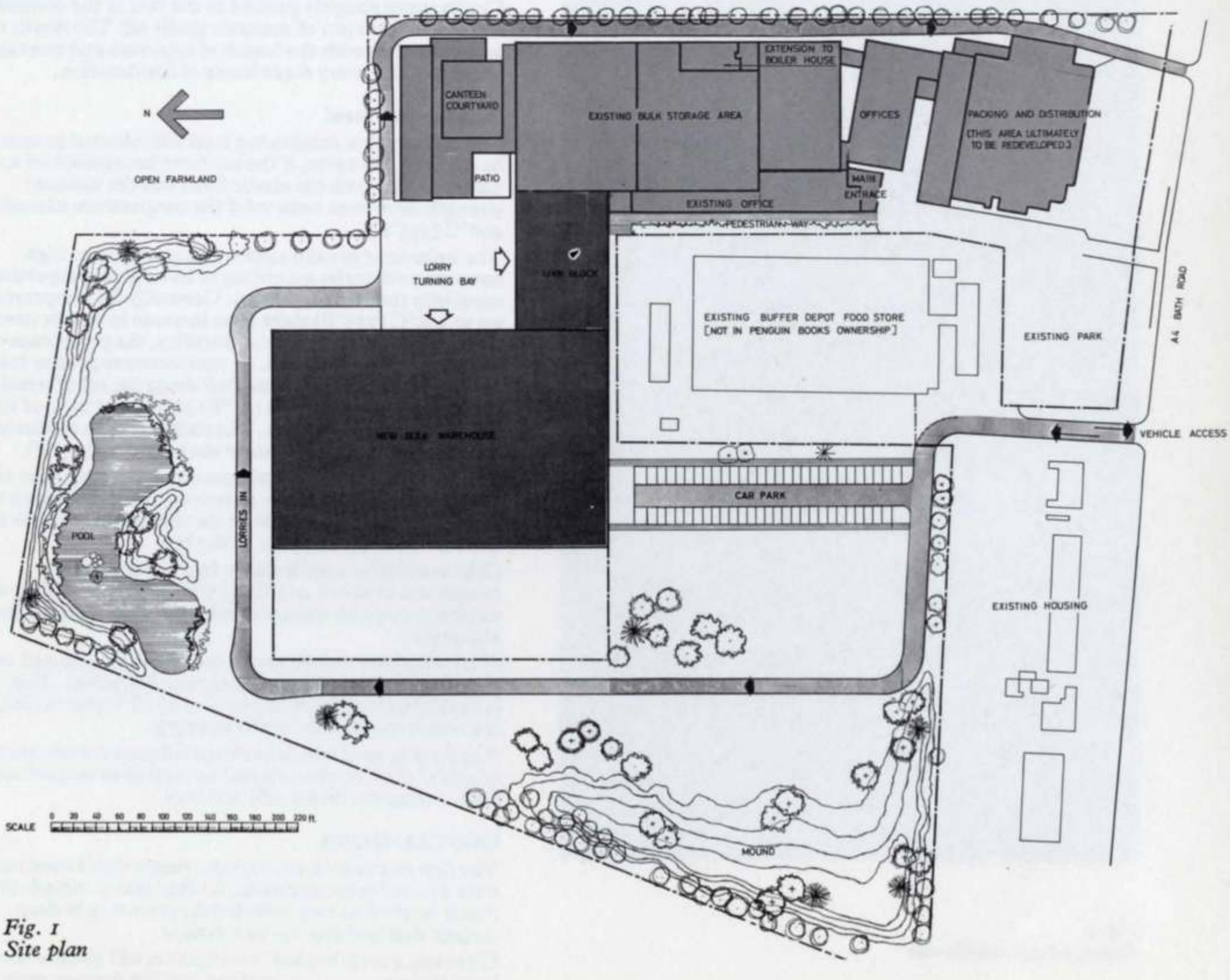


Fig. 1
Site plan

STRUCTURE

In a building of this type the structural module is a dominant feature.

Of the three dimensions to be determined, two were known—a lateral dimension of 13 ft. 6 in. or multiples of it and a clear height of 28 ft. The longitudinal column spacing would be determined by the economics of the structural span.

There were clearly two alternative structural media for this building, steel and precast concrete. Each have their own protagonists. Our clients had already been approached by Modern Concrete (Bristol) Ltd. with a view to the use of their precast hyperbolic paraboloids. However, the available sizes just did not suit the 13 ft. 6 in. module.

The problems of roof drainage, the client's requirement for a high degree of natural daylight and improved insurance rating of the Fire Officers' Committee were but a few of the factors which led to the present structural solution in concrete.

FOUNDATIONS

It was important that the floor level of the new warehouse should be the same as the existing building. In order to maintain this level, and to satisfy the floor loading of $\frac{1}{4}$-ton/sq. ft., it was inevitable that the site had to be stripped down to the ballast level and back-filled to the underside of the ground floor slab. A good 10 ft. deep strata of sandy gravel under the vegetable soil provided us with an excellent back-fill and by digging out a corner of the site we not only obtained a cheap back-fill but also formed the lake.

In order to satisfy the local authority, we agreed to have an engineer on site full-time taking density tests on this fill which we found consolidated to a maximum after four passes of a 10-ton roller. Surprisingly, the fill density proved to be some 10 lb/cubic ft. greater than in its natural state.

THE ROOF

A precast concrete 'V' shaped trough unit formed the basis of the roof structure. The units 9 ft. 6 in. wide, 4 ft. deep and $2\frac{1}{2}$ in. thick, formed a 300 ft. continuous beam, supported internally on columns at 77 ft. centres.

The beams were centred 13 ft. 6 in. apart which formed a 4 ft. continuous strip of daylight down the aisles.

ERECTION

The 24 in. x 12 in. internal columns and the 24 in. square external columns were erected by the post hole method which enabled a very high degree of accuracy to be obtained. The trough units were lowered onto preformed dowels on the head of the columns. The units were then jointed by means of a male and female keyed end diaphragm which was grouted and bolted.

EXTERNAL WALLS

We investigated the possibility of using insulated precast concrete faced planks which would satisfy the design requirements for the 28 ft. high wall. However, we found, in attempting to overcome insulation, waterproof joints and acceptable concrete finishes, that it was too expensive.

The cheaper solution was to slot 2 ft. deep x 4 in. thick reinforced precast concrete planks between the external columns. Galvanized cramps projected from a mortar bed to take an external skin of facing brickwork.

NATURAL ROOF LIGHTING

The natural roof lighting was achieved by bowing a corrugated sheet of wire reinforced pvc across the 4 ft. gap. This considerably increased its structural rigidity and

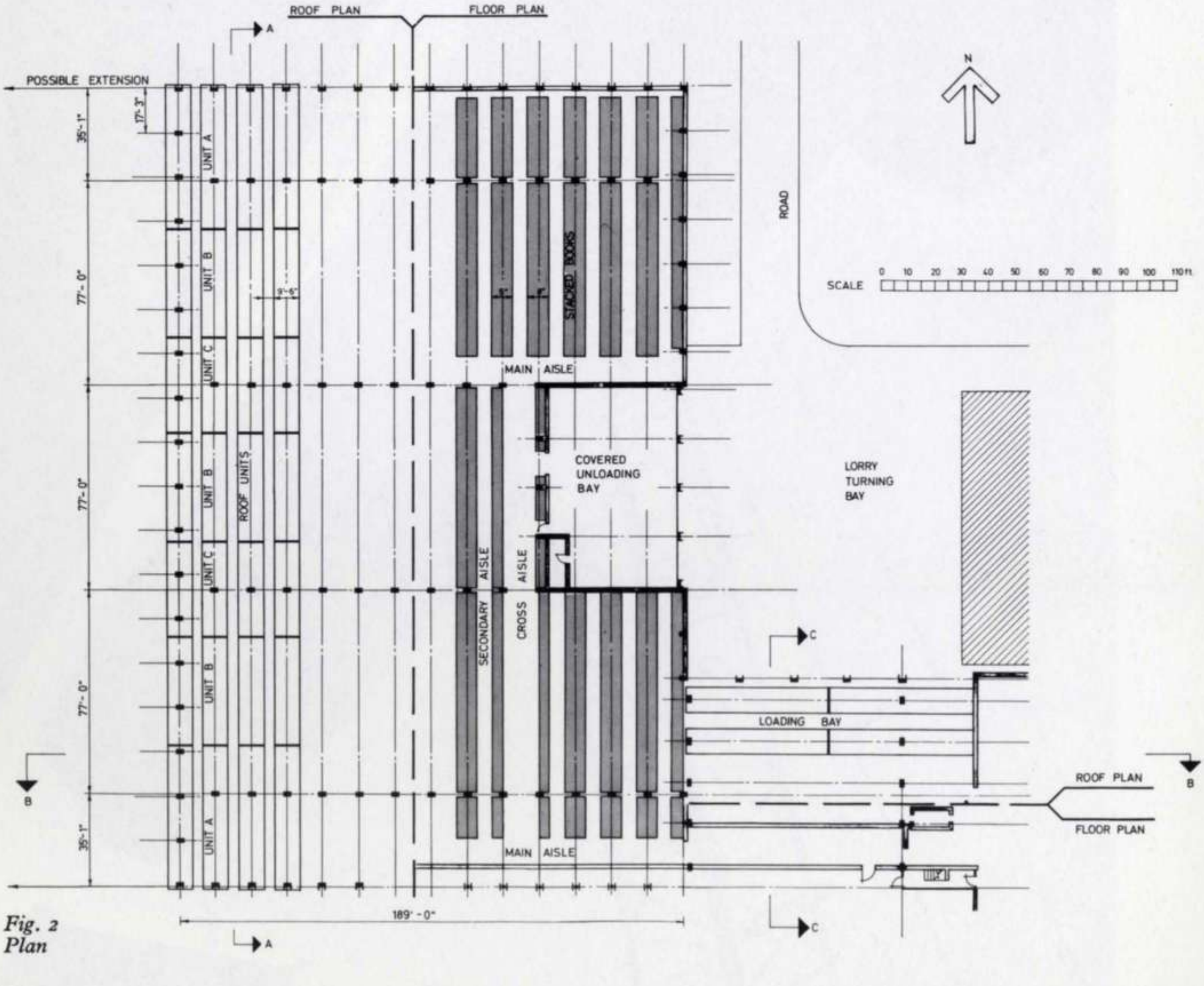


Fig. 2
Plan

loading capacity and, as this was a new technique with the material, the suppliers carried out a number of tests with us on edge buckling, eaves' sealing and fixing methods.

ROOF DRAINAGE

Roof drainage is taken down the 24 in. square columns at the north and south ends and discharged across depressions in the external pavement into open gutters which drain to the lake.

HEATING

Heating is provided by hot water batteries above the transverse 11 ft. wide main aisles at a level high enough to clear the lowered mast of the fork trucks. Heating is sufficient to maintain the internal temperature at 50°F with an external temperature of 32°F.

VENTILATION

Ventilation is provided by incoming air at low level through the honeycomb brickwork on the north and south elevations. Ventilation occurs by natural stack effect through ventilators in the 4 ft. wide roof glazing strips. The low level ventilators can be closed by horizontally hinged flap doors and the roof vents by cord operated louvres.

FIRE PROTECTION

The roof vents are also spring-loaded against fusable links for automatic smoke relief in the event of fire. Fire is further guarded against by a sprinkler system with heads which alternate between light fittings on either side of the roof units.

In addition to the main warehouse, we are carrying out work on a new boiler house in the existing building and we are installing a computer.

The building is due to be completed in August of this year.

Fig. 3
Erection of roof unit
(Photo: A. B. Gourlay)

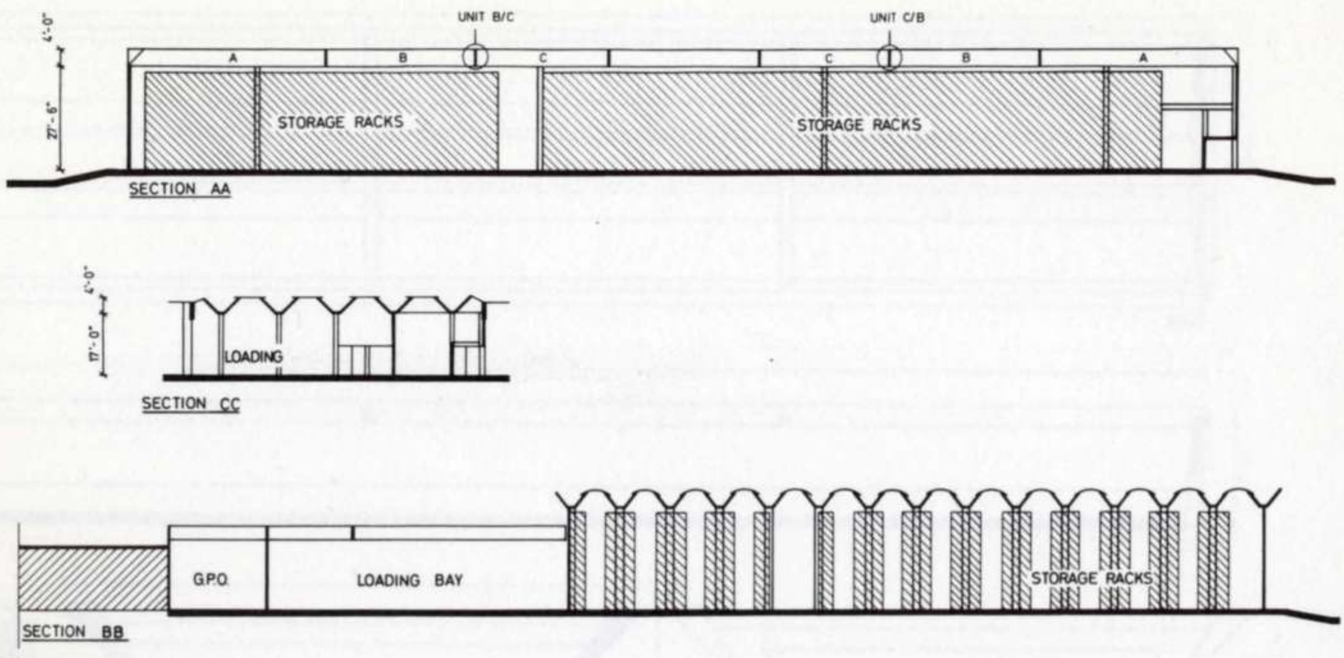


Fig. 4
Sections

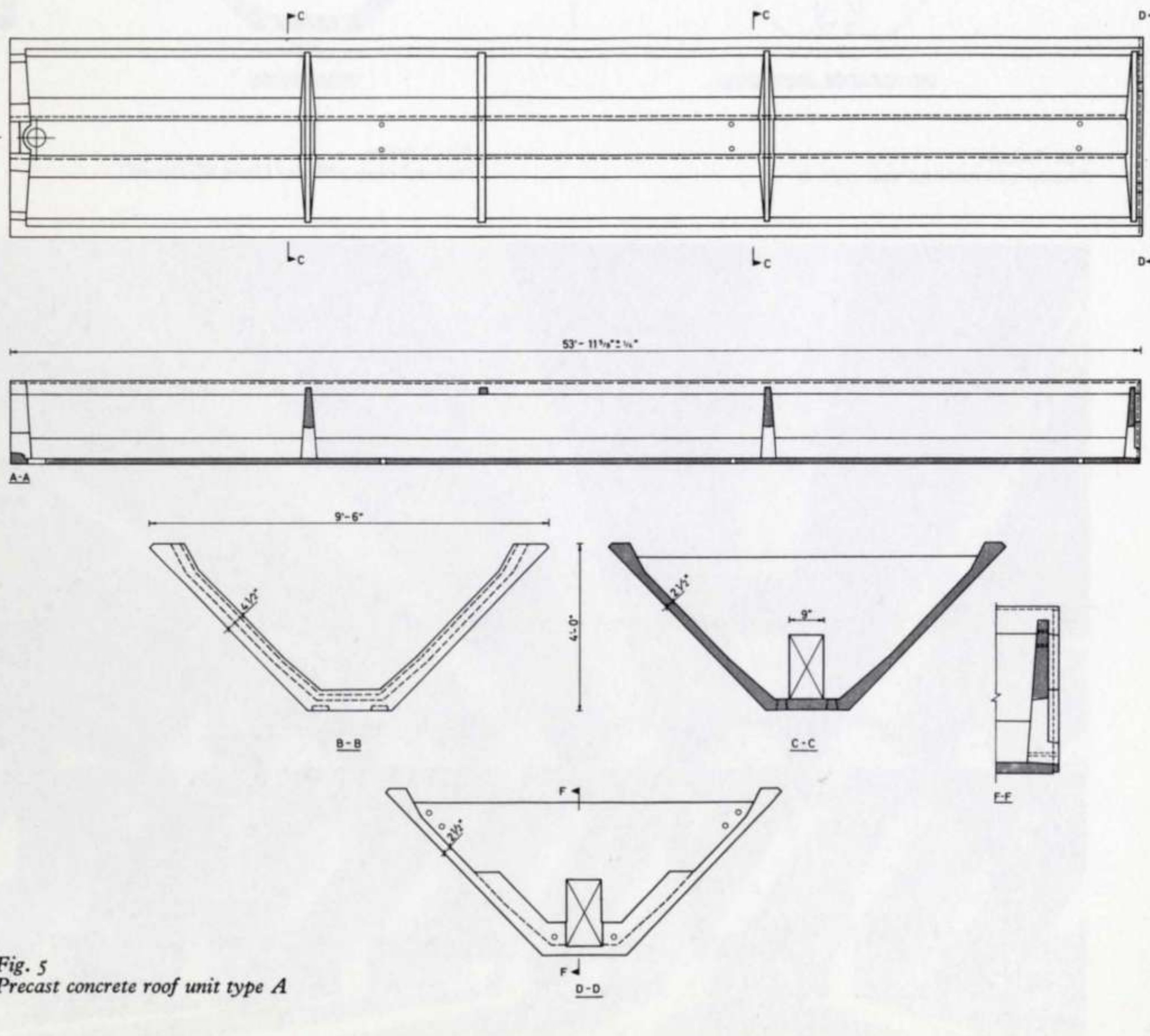


Fig. 5
Precast concrete roof unit type A

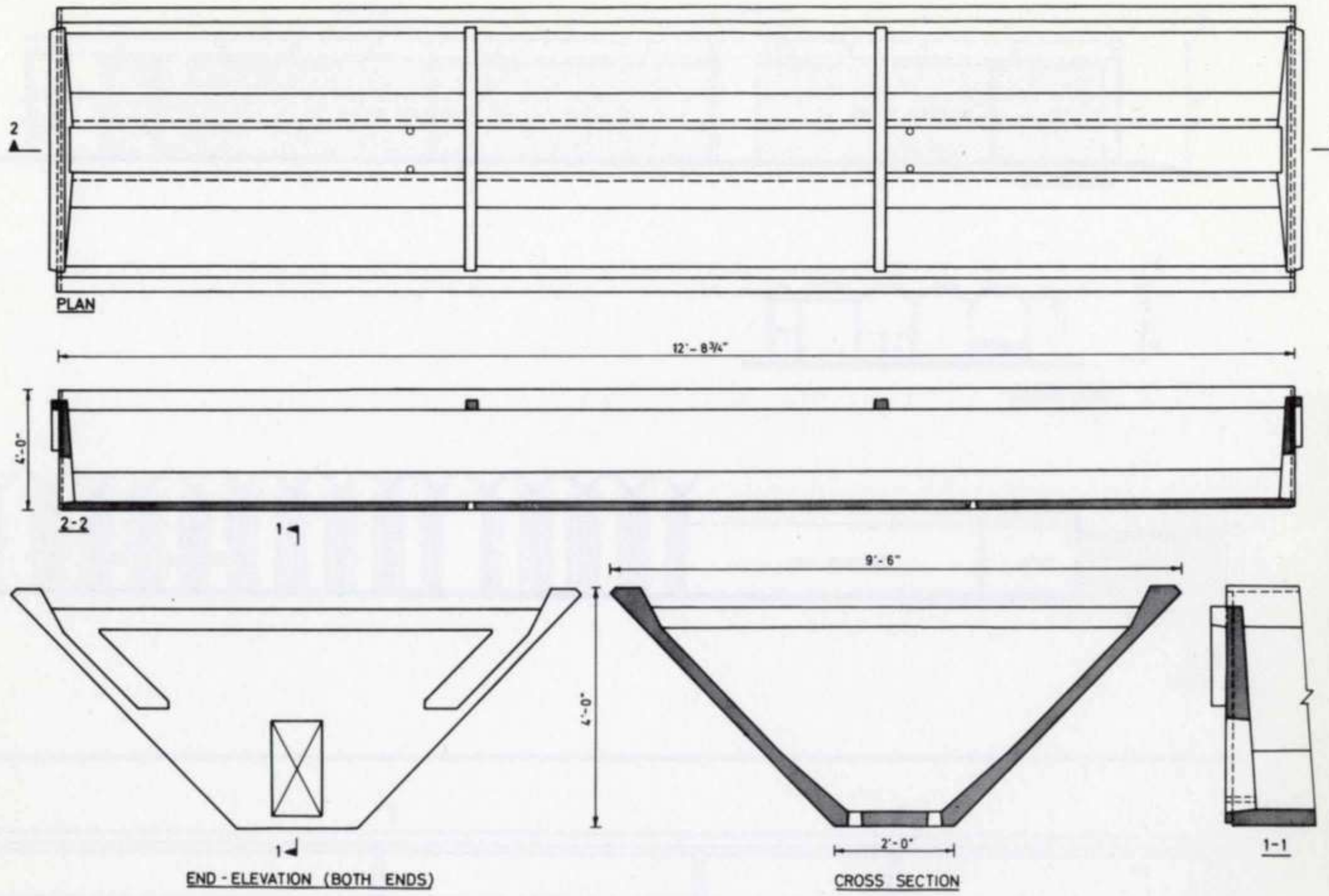


Fig. 6 above
Precast concrete roof unit type B

Fig. 7 below
Internal view (Photo: Colin Westwood)

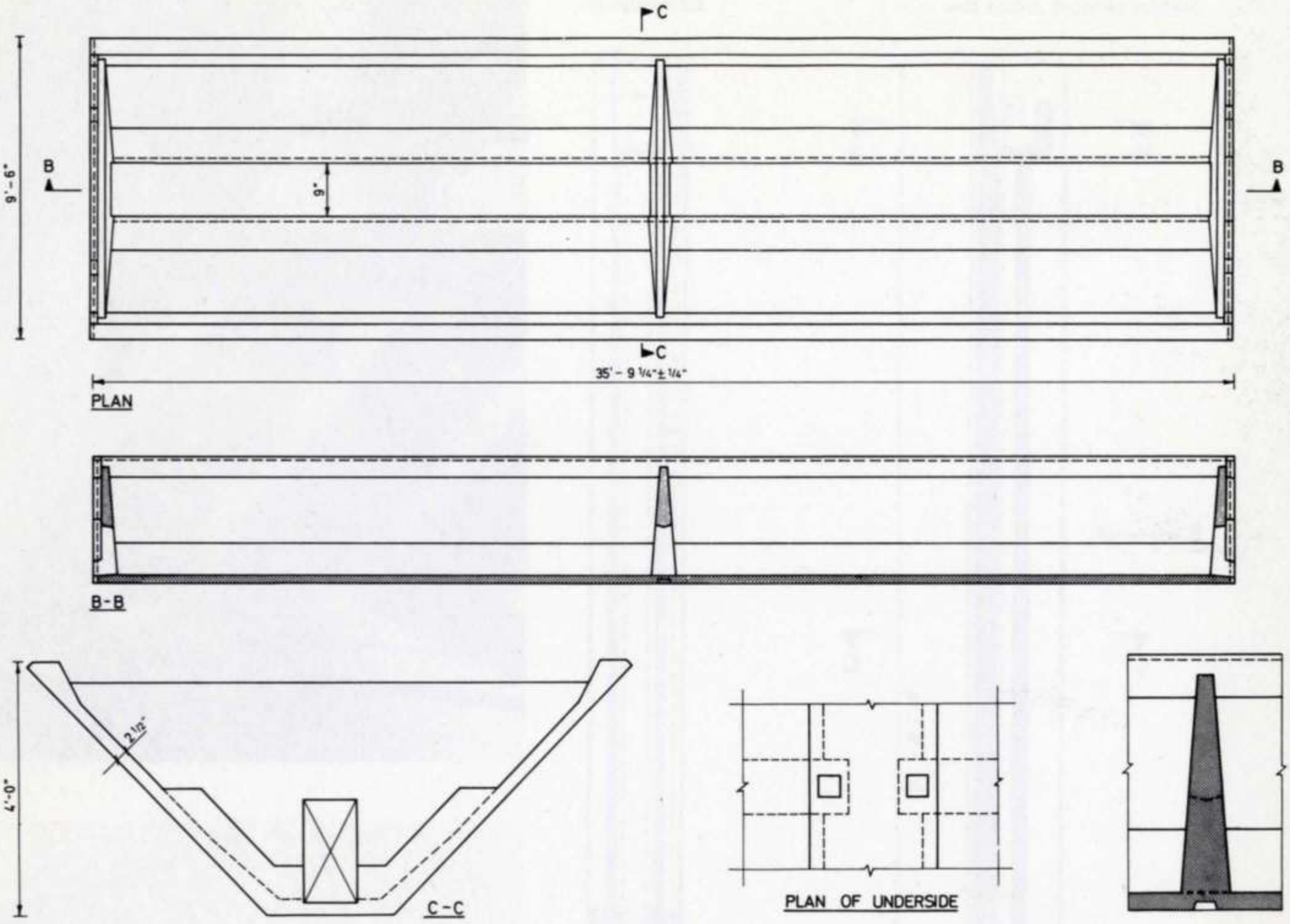
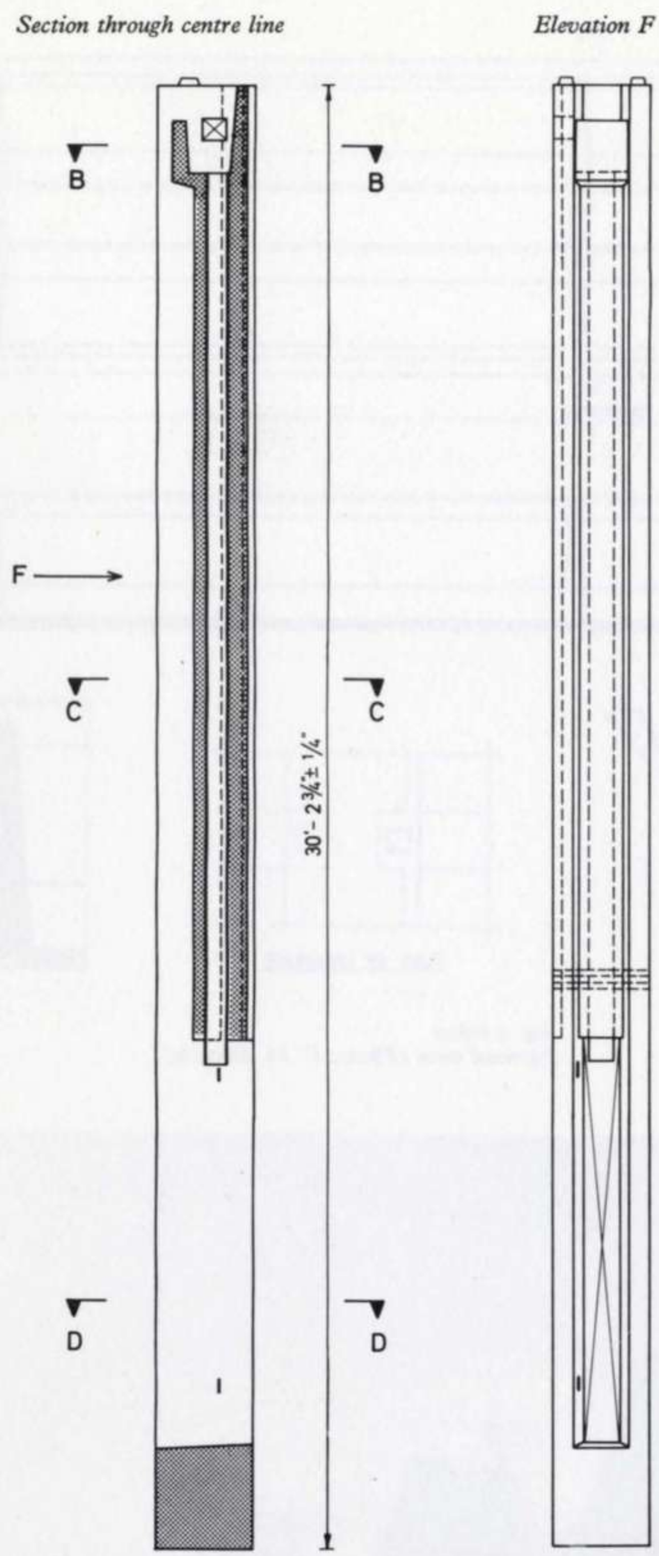
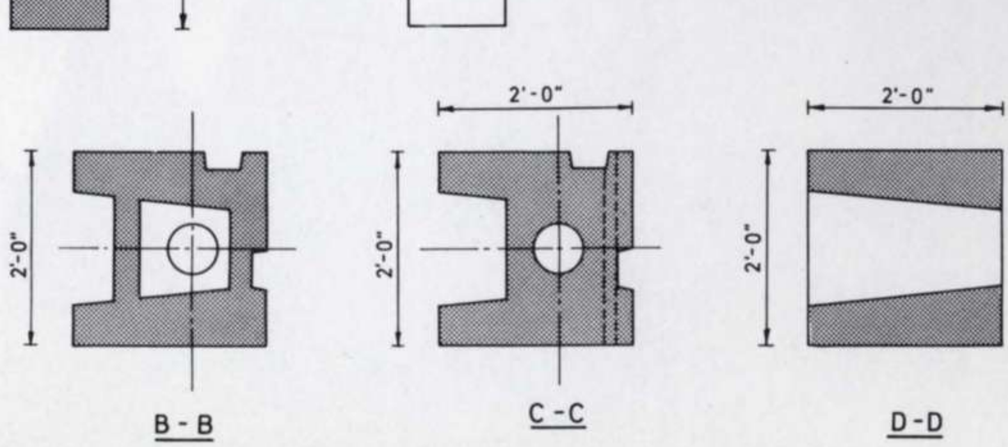


Fig. 8 above
Precast concrete roof unit type C

Fig. 9 below
External view (Photo: P. M. Dowson)





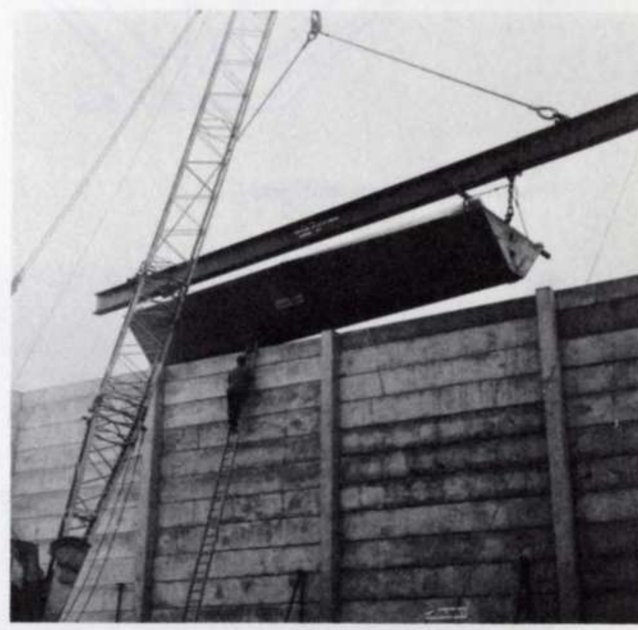


Fig. 10
Precast concrete external column (south)

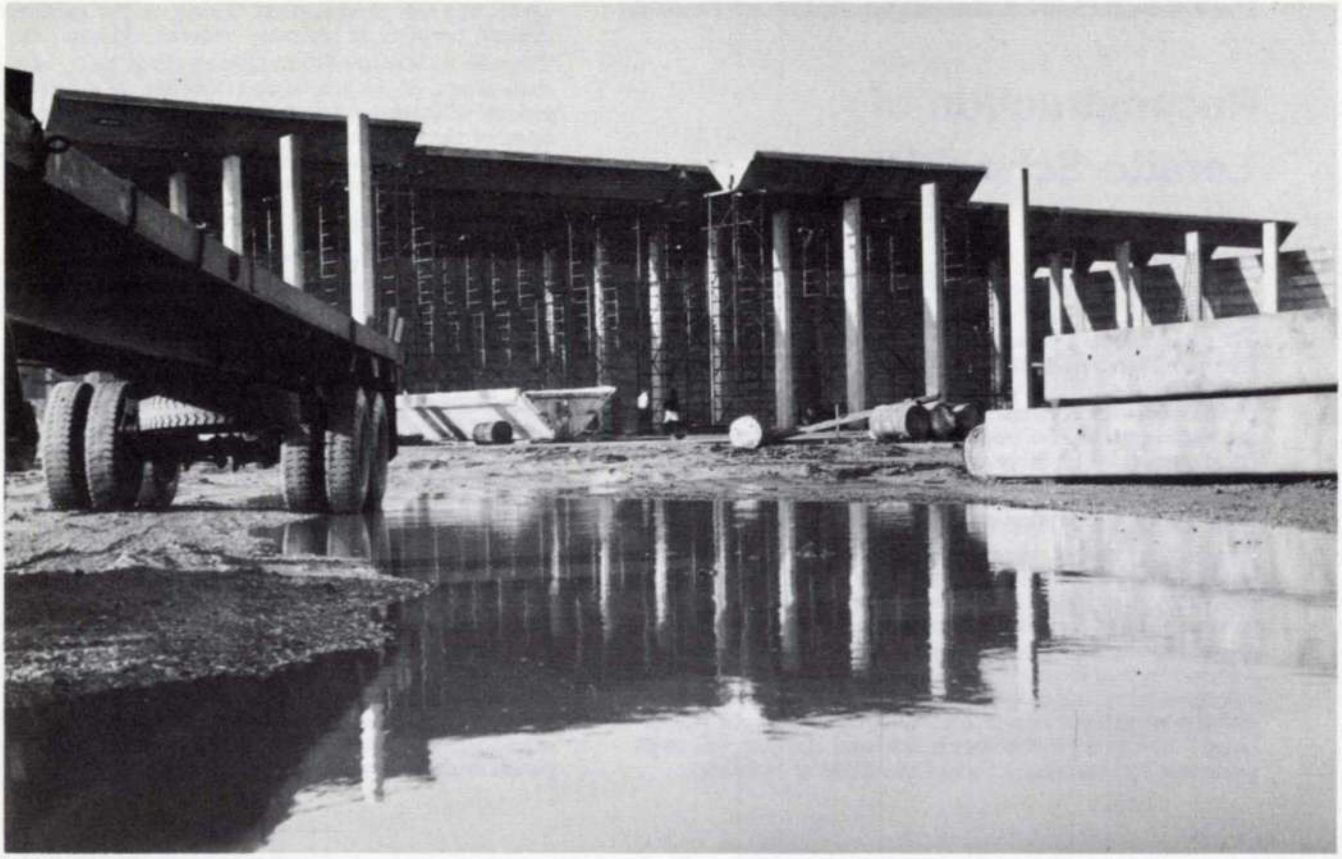


Fig. 11 top left
Internal view
Note method of ventilation control
(Photo: Colin Westwood)

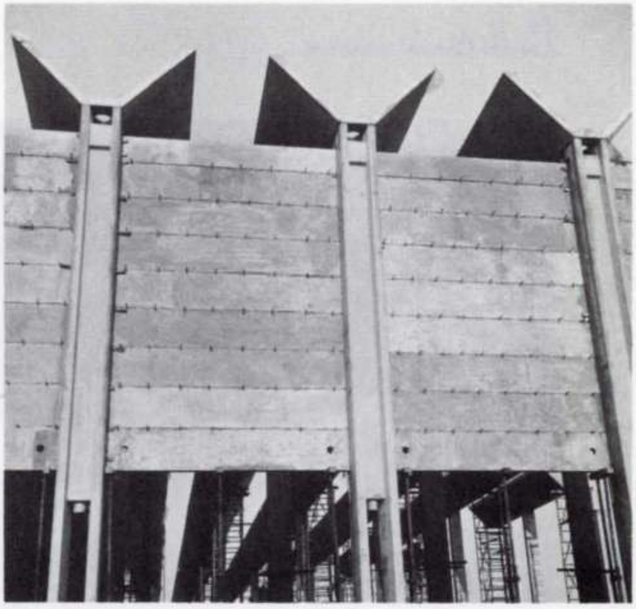
Fig. 12 left
Erection of roof unit
(Photo: A. B. Gourlay)

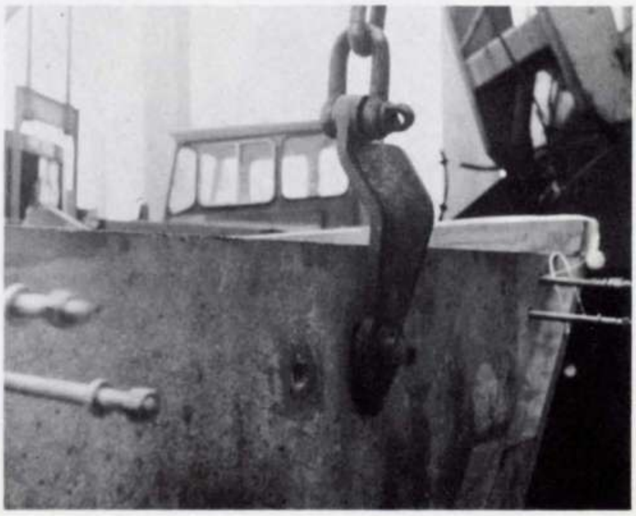
Fig. 13 above
External view during erection
(Photo: A. B. Gourlay)

Fig. 14 right
Precast concrete wall panel
between external columns
(Photo: A. B. Gourlay)

Fig. 15 below
Erection of wall panel
(Photo: A. B. Gourlay)

Fig. 16 below right
Lifting hook
on end diaphragm of roof unit
(Photo: A. B. Gourlay)





Reconstruction of Loretto School Chapel

J. Shipway

This was a little job in which we assisted Robert Matthew, Johnson-Marshall & Partners. An unusual feature was the construction of three large windows holding panels of multi-coloured cut glass. We strove valiantly to provide the architect with the exact effects he wanted and the efforts exercised us more than we thought possible on a job of this size.

THE OLD AND THE NEW

The original chapel was constructed about the year 1900 in the Gothic style, with arched timber collar beams supporting a steeply pitched roof. The building was 30 ft. wide and 90 ft. long, and the arched roof was approximately 45 ft. high at the apex. A gift of about £90,000 from a former pupil made possible the enlargement of the chapel from its original seating capacity of 370 persons to 480. This handsome gift was hedged by the condition that a link with the past was to be preserved by retaining a part of the existing building within the new construction. This gave the architect a difficult problem of harmony between old and new.

The scheme involved demolishing about 40 ft. of the chancel end of the chapel and building on a new wider section 50 ft. broad by 60 ft. long. The ridge line and roof slope of the old section were maintained and in place of the former arched collar beams, laminated timber portal frames spaced at 15 ft. centres were introduced in the new work.

THE WINDOWS

The largest window of the three was triangular in shape and formed the east gable of the enlarged chapel. Its lower edge was about 8 ft. 6 in. above the ground and it was 37 ft. high and 52 ft. wide at the base. It was to be constructed of concrete panels containing coloured cut glass—not stained glass—from Lyons, France, the work of John Lawrie, a lecturer at Edinburgh College of Art. The cut glass is of pure colour and is chipped so that the surfaces act as lenses, giving varied effects according to the light and the season of the year.

The panels were 24 in number and of various sizes, the largest being about 5 ft. x 3 ft. 9 in., and weighing 5 cwt. The glass was set in the concrete by John Lawrie, and this concrete was wire-brushed 6-12 hours after casting to produce the textured surface he desired to set off the glass. This same surface was to be reproduced exactly in the window mullions and frame. It soon became evident that, since John Lawrie preferred to work without assistance on the panels, they would be ready only just in time for the date of re-dedication of the chapel, and the window would therefore have to be designed and built to hold the panels without the advantage of having them cast in.

Fig. 1
Loretto Chapel extension.

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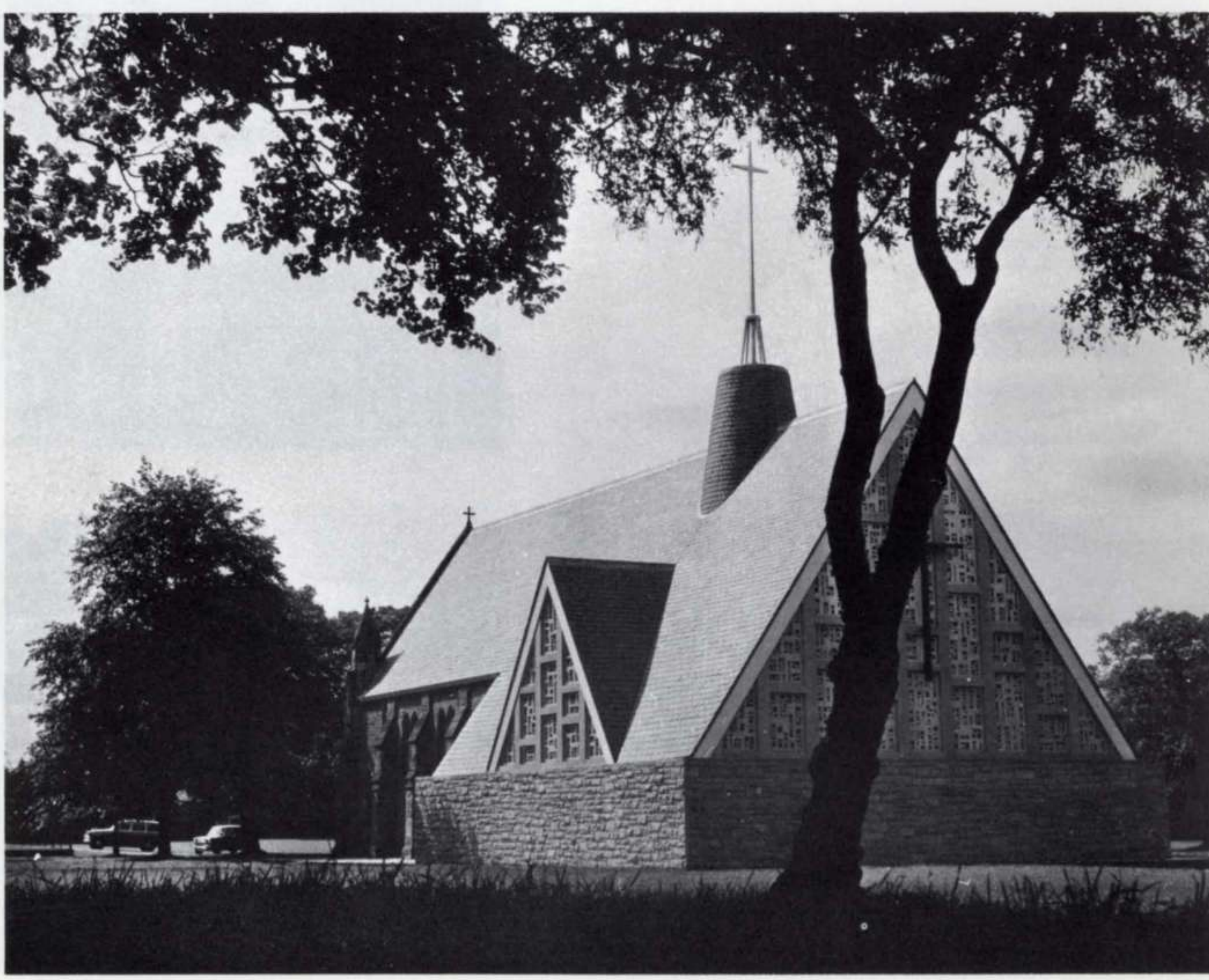


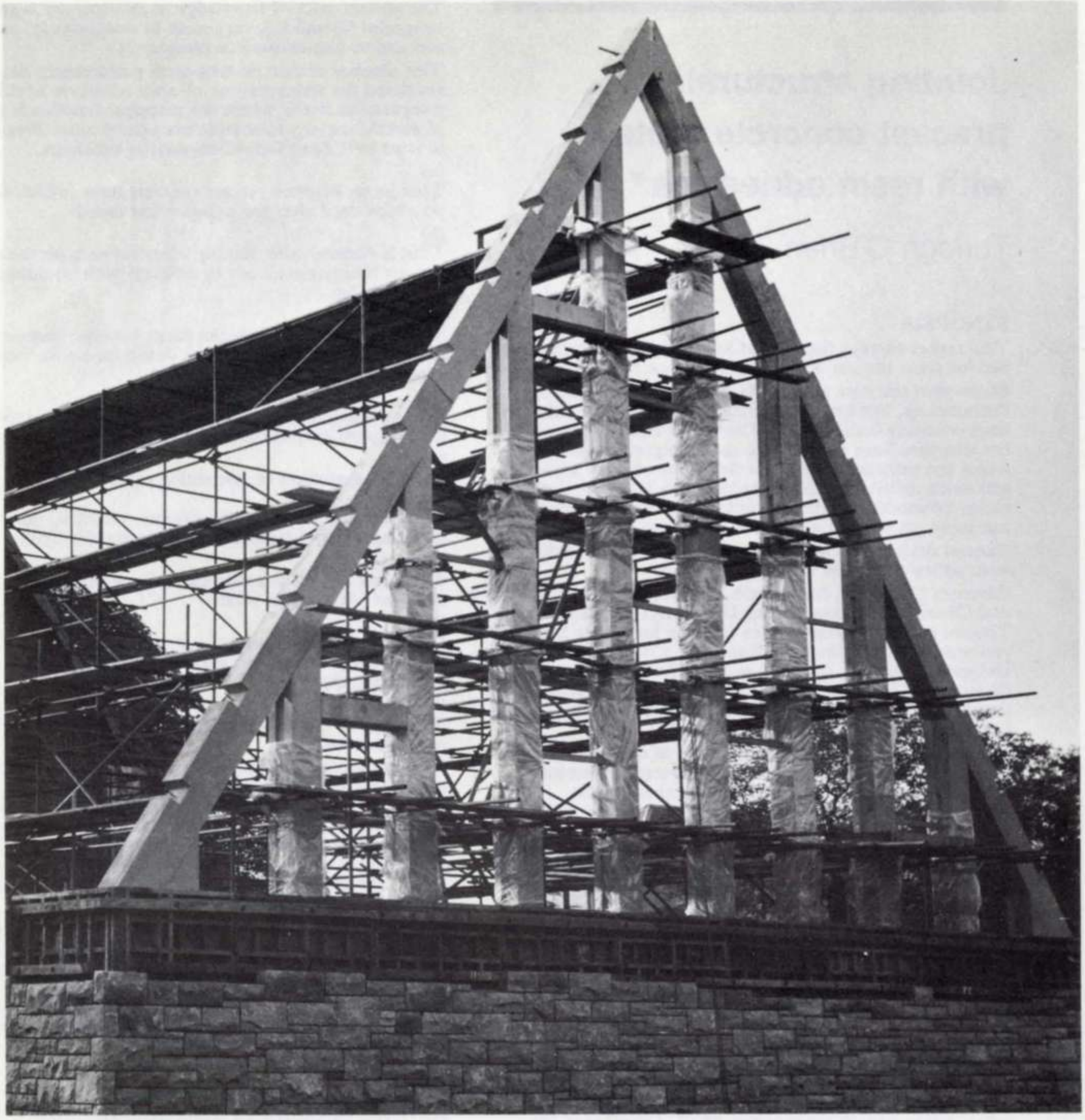


Fig. 2
Loretto Chapel extension

THE MULLIONS

The architect wanted the same finish on all surfaces of the mullions. The largest of these was 32 ft. long, 18 in. x 13½ in. in section, and weighed 3 tons. We suggested precasting mullions and sills and expressing the joints, but joints were declared unwanted. It was obviously impossible to pour the window vertically in situ without a construction joint, so we prepared a scheme for the window to be cast horizontally on a trestle strongback on the ground, raised through 90° to vertical using the strongback, and then wire-brushed on all faces 12 hours after casting. This scheme was completely detailed with contractor collaboration. However, after further discussion the scheme was abandoned and precasting mullions and sills with expressed joints was adopted.

Each mullion was anchored at the base into the 8 ft. 6 in. high stone clad concrete wall, and the whole window was calculated to be stable against wind effects by cantilevering vertically from the wall. The joints between mullions and rafter edge pieces were made by dowels grouted in, but the horizontal sill pieces were laid in position over 3 in. square pencils spanning between sockets in the mullions, both sills and pencils having dry joints.

The cut glass panels all arrived in time, and were inserted in the concrete frame and held by phosphor-bronze spring dowels engaging sockets in the mullions, rather like the ball-catch on a door. A feature of the spring dowels was that once in position and released, it was impossible to remove a panel without breaking it up. Fortunately this problem never arose.

THE CROSS

The new building is graced by a slender cross standing 34 ft. above the ridge line. The vertical member was designed of hollow timber section, and was waisted to 3 in. square at a distance of 8 ft. 6 in. below the top. These figures are in the realm of yacht-mast dimensions, and the cross was in fact manufactured by the well-known boat-building firm of McGruer & Co., Clynder, on the Clyde. The cross is covered with a fibre-glass sheathing coated with aluminium powder, as is the structural steel frame supporting its base, and the overall effect is of satin metal finish.

Photographs of the new chapel and window were exhibited at the Royal Scottish Academy Exhibition of 1965, and won favourable comment in the press.

Jointing structural precast concrete units with resin adhesives*

Turlogh O'Brien

SYNOPSIS

This report surveys the state of knowledge and experience derived from the use of resin adhesives on twelve buildings.

Short-term test data are readily available for a variety of formulations, but long-term mechanical behaviour has been relatively little studied. The use of resin adhesives has therefore been restricted to thin compression joints, where the principal function is that of an efficient gap-filler and stress distributor. It has been found, however, that under favourable weather conditions the rate of hardening can speed up the rate of erection of precast concrete units.

General descriptions are given of the joints formed with resin adhesives on the following buildings:

Coventry Cathedral; Abbotsinch Airport; Physics building and Chemistry building Exeter University; Walton-on-Thames swimming pool; Office building for Evode Ltd.; two residential buildings for Somerville College, Oxford University; Mining, minerals and metallurgy department, Birmingham University; New Museums laboratories, Cambridge University; Netheredge Maternity unit, Sheffield; Sydney Opera House, Australia.

The second half of the report describes in detail the use of epoxy resins for structural jointing between the precast segments of the ribs that form the roof shells of the Sydney Opera House. The preliminary laboratory testing, the surface preparation, the application technique, the control testing and repairing of cracked joints are described. An assessment is also given of the advantages gained by use of this jointing technique. Mention is made of the miscellaneous other uses to which epoxy resins have been put on this building.

The report, which is fully illustrated, concludes with a review of the deficiencies in knowledge, which are holding up the development of structural glueing.

ACKNOWLEDGEMENTS

The designers of the buildings described are named in the text. Where only the architect is given the consulting engineers were Ove Arup & Partners. The method of construction of the Sydney Opera House was devised in collaboration with the contractor, M. R. Hornibrook Pty. Ltd. of Sydney. Dr. John Nutt of Ove Arup & Partners, Sydney, assisted with the writing of the section describing the Opera House.

INTRODUCTION

For any new jointing compound to be used widely with confidence a detailed knowledge of its mechanical behaviour is required. For resin adhesive materials five categories of behaviour may be distinguished.

1.
The mechanical properties of the adhesive material under short-term testing.

2.
The bond strength to particular materials under short-term testing.

3.
Fire resistance data of various typical joint details.

4.
Long-term deflections under various states of stress.

5.
Long-term strengths of the adhesive in various enviroments.

The present state of knowledge in this country is good in categories (1) and (2), very weak in categories (3) and (4) and almost non-existent in category (5).

This absence of data on long-term performance has restricted the structural use of resin adhesives to thin compression joints, where the principal function is that of an efficient gap-filler and stress-distributor. Five types of joint have been formed on various buildings.

(a)
Thin joints between precast concrete units, which are post-tensioned after the adhesive has cured.

(b)
Thin horizontal compression joints between sections of precast concrete mullions or columns with no dowel connector bar.

(c)
Thin horizontal compression joints between precast concrete column sections, with dowel connector bars grouted in with the same adhesive.

(d)
Precast concrete shear connections using bolts to maintain the adhesive in compression.

(e)
Various joints used in assembling precast concrete staircases.

In addition to the stress-distributing function, other advantages may be obtained from resin adhesives.

(i)
The joints appear as a thin line instead of the ½ in. band obtained with mortar joints.

(ii)
Under favourable weather conditions the rate of hardening of the adhesive can assist early de-propping, post-tensioning, etc.

(iii)
Shrinkage in the joint is negligible, leading to greater watertightness.

The first section of this paper demonstrates how these advantages have been realised in joints of the types described. The buildings referred to have either been completed or are being constructed, with the glueing already finished. The second section describes in detail the use of epoxy resins on the Sydney Opera House. The final section discusses the further work that is indicated from experience on actual buildings.

It may be noted that two of the buildings referred to have used polyester resin adhesives instead of epoxies. They have, however, been used in a manner very similar to the epoxies, and although much less is known about their long-term performance, they appear to be more tolerant of site conditions than epoxies. This, coupled with their lower cost, is a significant advantage.

SOME BUILDINGS INCORPORATING GLUED JOINTS

Coventry Cathedral—Architect: Sir Basil Spence, O.M., R.A.—1960-62*

The slender cruciform columns that support the inner ceiling of slatted spruce panels (Fig. 1) were precast in three sections of equal weight. These sections were stressed together by four stranded cables. A jointing compound was required which would provide uniform transfer of compressive stresses, and be almost invisible. An *Araldite* epoxy resin from CIBA (ARL) Ltd. was used.

In the chapel, which is circular in plan, there are slender precast concrete mullions. The three sections were jointed with the same *Araldite* resin, but in this case there is no continuous steel across the joint. As with the column joints the thickness was required to be a minimum, as they are visible internally, though slate-clad externally. The joints act purely in compression with working stress of 600 lbf/sq. in.

The units were mated on site to ensure complete match and alignment at the joints. This was achieved by first lifting the units into position, where they were held horizontally with templates, but with wooden wedges keeping the joints open. When a joint was to be mated the unit was raised and lowered with a small hand hoist,

Editorial note: Paper presented at Rilem Symposium on experimental research on the new developments brought by synthetic resins to the techniques of concrete, reinforced concrete and masonry. Paris 4-6 September 1967.

Editorial note: The dates given are when the glueing was done.

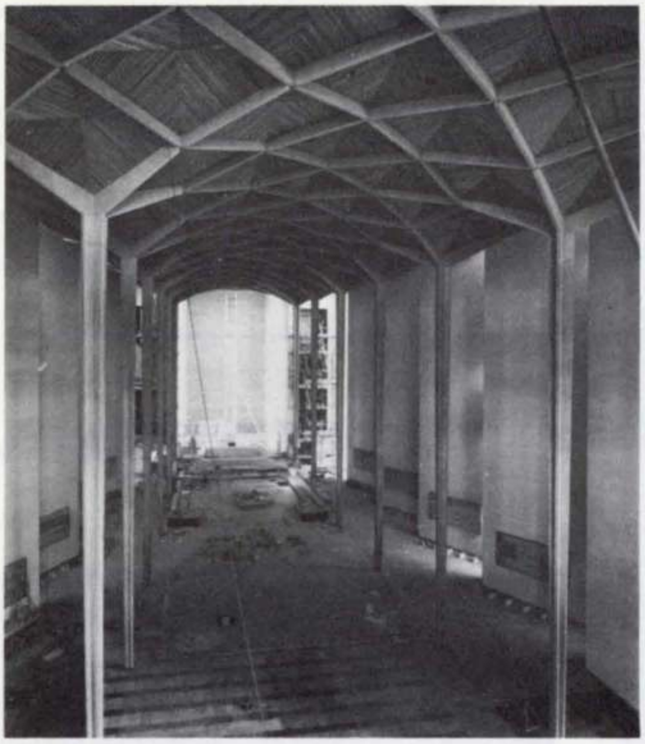


Fig. 1 above
Interior view of Coventry Cathedral nearing completion showing the columns and ceiling. The columns were precast in three sections. (Photo: John Laing & Son Ltd.)

Fig. 2 below
Interior view of the Guild Chapel at Coventry Cathedral, showing the slender mullions. (Photo: John Laing & Son Ltd.)

imperfections in the surfaces and the exposed ends of the reinforcing bars were removed by grinding. When a perfect match was achieved, the surfaces were scrubbed with a solution of *Teepol* and washed with clean hot water.

On complete drying the edges of each unit were taped to protect them from overspill. A CIBA *Araldite* resin was mixed in sufficient quantity for one joint at a time. It was applied by trowel to both surfaces. After lowering and aligning the top unit, the curing pressure was provided by the self weight of the unit.

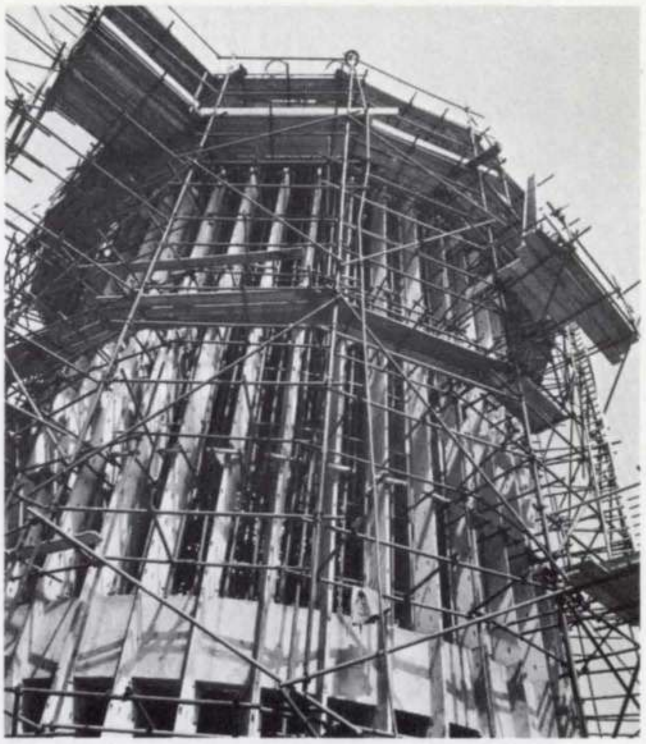
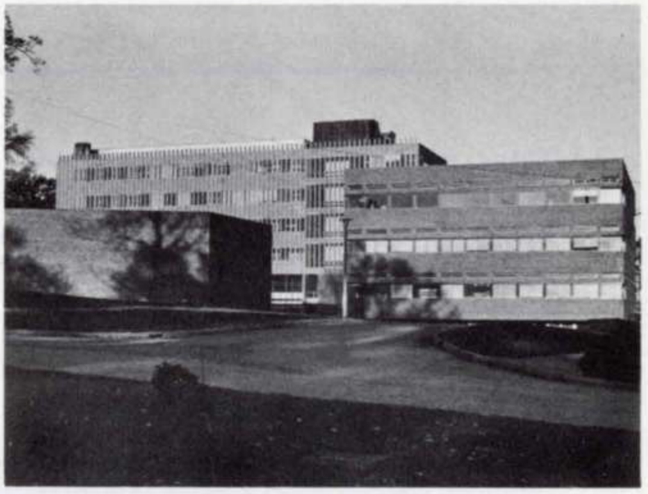


Fig. 3 above
View of the Guild Chapel at Coventry Cathedral under construction. (Photo: John Laing & Son Ltd.)

Fig. 4 below
The research wing of the Chemistry building at Exeter University. The slender mullions were cast in two sections. (Photo: E. Happold)



Exeter University Science Buildings—Architects: Sir Basil Spence, Bonnington & Collins—1964, 1966

The new Chemistry and Physics Buildings at Exeter University have some slender precast concrete mullion units, which are similar to those supporting the roof of the Guild Chapel at Coventry Cathedral. The units are about 26 ft. long with a cross-section shaped like a blunt arrow-head. The cross length is 21 in. and the thickness varies from 2 in. to $4\frac{1}{2}$ in. to $2\frac{1}{2}$ in. The cross-sectional area is 62 sq. in. Each unit is two floor heights in length.

In the Research Wing of the Chemistry Building, the units are at 2 ft. centres along either side of the building and support three floors and the roof in two lifts. The floor slabs span onto cross-beams which, in turn, span into the edge-beams which deliver the load to the units.

In the Tower Block of the Physics Building the units are at 2 ft. centres around the entire perimeter and support five floors and the roof in three lifts. These floors span directly into the edge-beams. The units are of uniform section throughout their length, with holes left in them for passing through the edge-beam reinforcement.

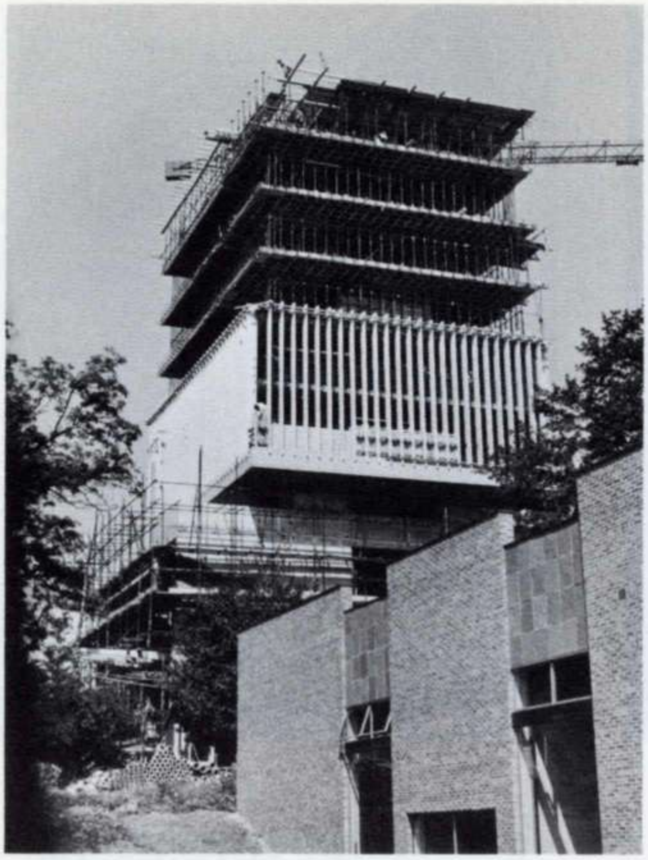


Fig. 5
The tower block of the Physics building at Exeter University, showing the first set of mullions in place, ready for the next set to be glued on. There will be three sections in the full height. (Photo: E. Happold)

The maximum compressive stresses in the units are 350 lbf/sq. in. for the Chemistry building and 1500 lbf/sq. in. for the Physics building.

In both buildings it was required that the joint thickness be kept to a minimum. For this reason, and for efficient load transfer, epoxy resin adhesives were used. The technique used was the same as that described for Coventry Cathedral. The materials used were CIBA *Araldite* AV 123B with Hardener HV 953B.

Abbotsinch Airport, Glasgow—Architects: Sir Basil Spence, Glover and Ferguson—1964

The roof structure of the terminal building consists of prestressed concrete trusses spanning between the main columns and carrying on both upper and lower booms precast concrete shell units. In each bay there is a 50 ft. truss in the centre and another on each side which spans 30 ft. and cantilevers a further 15 ft. The shell units span 20 ft. between trusses.

Each truss consists of a twin upper boom and a twin lower boom connected by a series of single members forming a Warren girder. In view of the difficulties of handling the slender 50 ft. booms, it was decided to cast them in 10 ft. lengths. The architect required very thin joints in the visible lower boom and these were made with epoxy resin adhesive. Other joints were made with ¾ in. dry packed mortar. The maximum stress in a boom is 1200 lbf/sq. in.

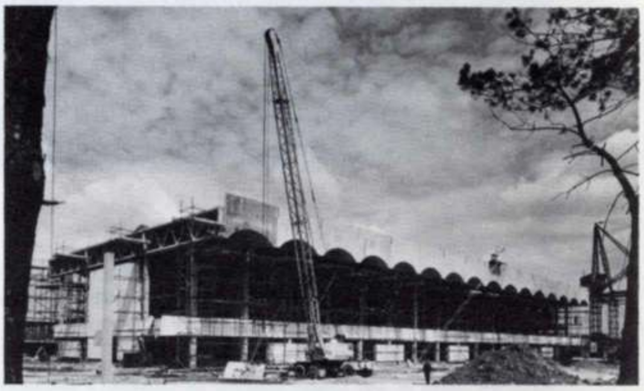
The units forming the bottom boom were cast against each other with a bond break between, a procedure described more fully for the Sydney Opera House. Stainless steel pins were cast in to provide accurate alignment. Rubber rings were used to prevent epoxy resin leakage into the ducts. The adhesive used was based on Shell *Epikote* 828 with a resin:hardener:filler ratio of 6:1:24. Light compression across the joints during curing was achieved by stressing one of the wires in the boom.

The use of epoxy resins in this application was generally successful. In two respects more thought would be needed in a repetition of this design:

(a)
The design of the casting beds and handling of precast units to achieve accuracy of matching faces for thin glued joints, and

(b)
The need for accurate jigs to bring the units together again in correct alignment prior to glueing, as the thin joints do not allow misalignments to be easily rectified.

Fig. 6
A construction picture of the terminal building at Abbotsinch Airport, Glasgow. The lower boom of the precast concrete roof trusses incorporates glued joints. (Photo: Studio Swain)



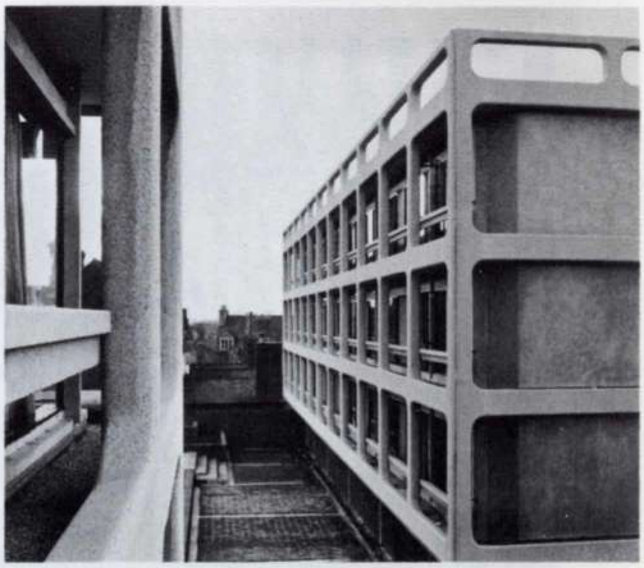


Fig. 7
The residential building at Somerville College, Oxford University, Phase II, in which the precast storey height external columns were joined with an epoxy adhesive. (Photo: Colin Westwood)

Somerville College, Oxford University—Designers: Arup Associates—1965-66

The initial concept of the Somerville College development assumed an in situ, white, bush-hammered external concrete frame. When the time of construction came, a precast frame proved more economical and phase I was built with 3-storey height columns, with in situ joints between beams and columns. Site restrictions for phase II, required the use of single storey height columns.

Fig. 8
Forming the joint between precast columns at Somerville College, Oxford University, Phase III. (Photo: Oliver Spence.)

Fig. 9
View of the frame of the Phase III building at Somerville College, Oxford University. (Photo: Oliver Spence)

The use of an in situ joint similar to the beam-to-column joint would have reduced the speed of erection, and as a self-supporting and aligning method was preferable, it was decided to use an epoxy joint. The column sections being dowelled together, it was only necessary for the epoxy joint to transfer compressive stresses. These are normally 1100 lbf/sq. in. with maximum value of 1400 lbf/sq. in.

As the final finish was to be equivalent to an in situ bush-hammered surface, the epoxy joint had to be of minimum thickness. This requirement did not allow for any joint tolerance to be taken up by packing and shimming. To overcome this problem the column was cast in a 3 storey height mould with $\frac{1}{4}$ in. mild steel dividing plates at each joint. This gave smooth matching faces for glueing.

The only problem during glueing was the clamping of the column sections. The resin acted as an efficient lubricant until set. CIBA's *Araldite* 123B with Hardener HV 953B was used and the result was reasonably satisfactory.

A similar solution has been adopted on the phase III building at Somerville. The column-to-column joints have a dowel connector, and speed of erection was an important consideration. The adhesive used was *Nitoflor Jointing Compound* from Chemical Building Products Ltd., a material based on CIBA *Araldite* resins.

This type of joint will be used again for the joints between the precast H-frame units of the Marist Technical High School, Manchester (Architects: Burles, Newton & Partners.)

Office Building at Stafford—Designers: Arup Associates—1963

The structure of this building included bolted connections between precast concrete edge-beams and columns. The dead weight of the beam is carried on a nib projecting from the column. To ensure uniform transfer of stress across the joint after bolting, an epoxy adhesive was used to cover the joint faces. This job was treated as experimental by the clients, Evode Ltd., who prepared and applied an epoxy resin adhesive of their own.

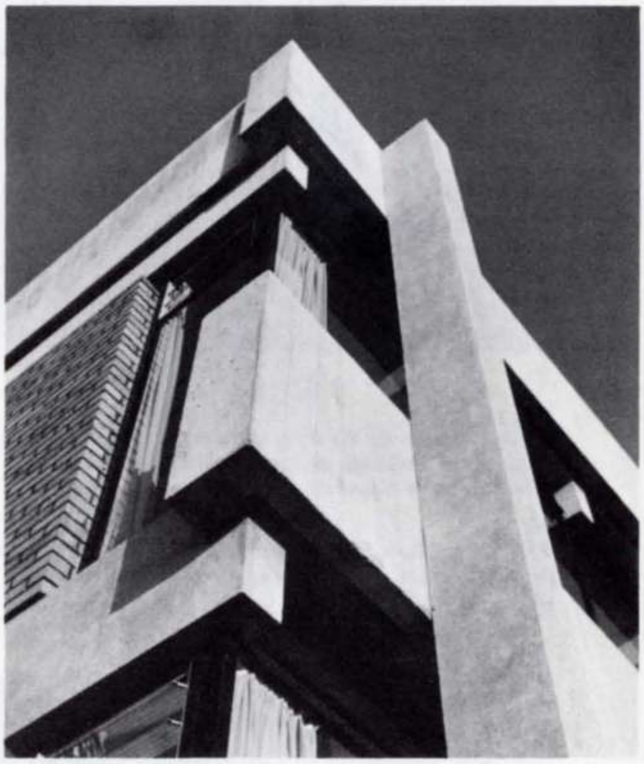


Fig. 10
Corner detail showing the beam to column joint on the office building at Stafford for Evode Ltd. (Photo: Colin Westwood)

Fig. 11
Swimming Pool, Walton-upon-Thames, showing the twin beams which are bolted to the columns with glued interfaces. (Photo: Colin Westwood)

Swimming Pool, Walton-upon-Thames—Designers: Arup Associates—1964

This building also incorporates bolted beam to column joints. In this case twin beams are bolted with stainless steel bolts to the columns. The contact faces were treated with an epoxy resin adhesive to give improved stress distribution. In this case there are no nibs supporting the beams and the adhesive acts in shear. The adhesive used was *Colma-Fix* from Sika Ltd. and the nominal glue-line thickness was $\frac{1}{8}$ in.

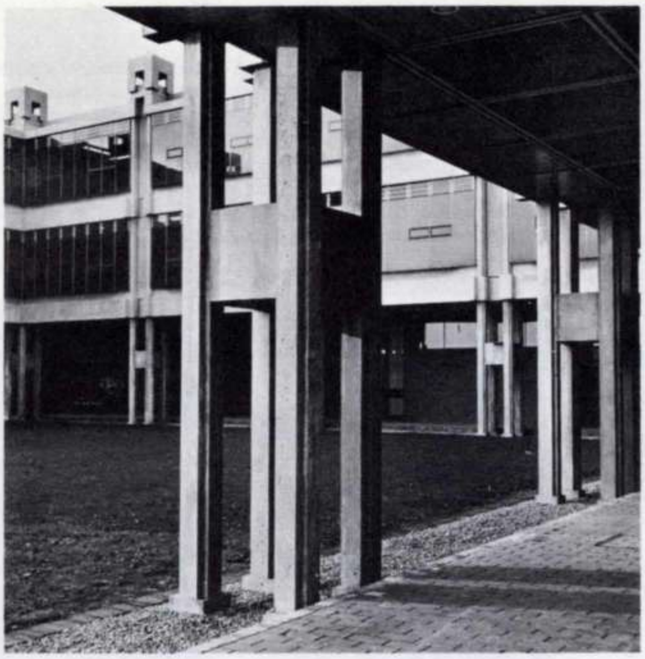
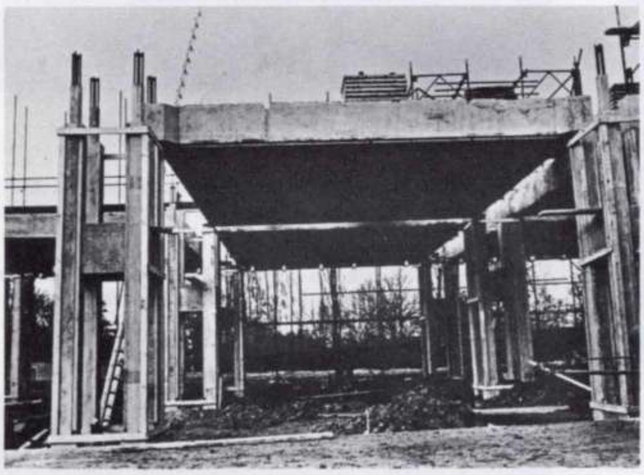


Fig. 12
Column clusters at the Mining, Minerals and Metallurgy Department buildings at Birmingham University.
(Photo: Harry Sowden)

Fig. 13
The Birmingham University buildings under construction, showing method of assembly of the precast components.
(Photo: John Hopkins)



Mining, Minerals and Metallurgy Departments, University of Birmingham—Designers: Arup Associates—1965

The outcome of the design process for this complex laboratory building, involving the interrelated considerations of planning, servicing and construction, was a structure consisting of a series of independent three storey towers 20 ft. square. The 2 ft. thick overall coffered floor and roof units are supported by a column at each corner. The towers are at 22 ft. 9 in. centres and the gaps between form the service ducts. Between every four adjacent corner columns is a vertical duct which intercepts the horizontal ones between the slabs. These slabs weighed 17½ tons each.

The superstructure was entirely precast. The columns had concrete spigots at the top which mated with sockets at the corners of the floor slabs. During erection the shoulders of the columns were buttered with a polyester resin adhesive, *Certite* from Stuart B. Dickens Ltd., before each slab was guided and lowered over the spigots. When the adhesive had set, the annular spaces between column spigots and slab sockets were grouted up to form a monolithic joint. Erection of columns on completed slabs followed a similar procedure. Reinforcing bars projected from the top of the column spigot below. The next column containing a socket, was placed over these bars and bedded with more *Certite* putty. Finally, the column sockets were filled with a fine concrete.

The function of the resin adhesive in this job was to act as a bedding material and grout seal. In this respect it was more efficient than a conventional mortar both in terms of load transfer and speed of construction.

The staircases in this building also incorporated *Certite* as a jointing material, where precast concrete flights were stressed against the spine with high strength Lee-McCall bars.

Netheredge Hospital, Maternity Unit, Sheffield—Architects: Morrison & Partners—1966-67

Epoxy resins have been used in this building in a manner similar to the Birmingham building. The precast concrete frame uses the adhesive as a bedding material where the beams rest on the columns. It also acts as a grout seal. The adhesive used was *Gircofix* from the Greengate Rubber Co.

This material was also used to bond precast concrete stair-treads to a precast structural spine beam. The units are 4 ft. 6 in. wide and oversail the spine by 8 in. on each side. They are designed to carry a maximum load of 300 lbs. at the edge of the unit. Mechanical fixing devices are eliminated by glueing.

Previously *Certite* had been used as a bedding between the precast units of a spiral staircase at Castle Market, Sheffield (1961).

New Museums Laboratories, Cambridge University—Designers: Arup Associates—1967

The design of this building includes two types of connections using epoxy resin adhesives. In one a joint is formed between precast concrete beams, which meet at right angles, by bolting a steel angle bracket to the beams. The efficiency of stress transfer from beam to bracket to beam will be improved by treating the interfaces with an epoxy resin adhesive.

In the administration towers, it is necessary to transfer the vertical loads from precast concrete walls to columns through a shear joint. Bolted connections are to be used. The bearing area of each joint is 2 ft. x 15 ft., and there are 28 bolts. The use of a nominal $\frac{1}{8}$ in. layer of epoxy resin adhesive will enable the shear stresses to be more uniformly distributed. The stress will be about 100 lbf/sq. in.

Sydney Opera House—Architect: Jørn Utzon—1963-1967

Description of the shell roof

The shells of the Sydney Opera House are formed by curved hollow concrete ribs which lie side by side so that a continuous spherical surface is formed. The ribs are made of precast segments, cast with matching faces, and made longitudinally continuous by post-tensioning across transverse epoxy joints. The segments weigh about 10 tons and vary in width from 1 ft. to 12 ft. and in depth from 4 ft. to 7 ft. The epoxy joints are a nominal 1/16 in. thick—in practice this varies from 1/64 in. to 3/32 in.—and a prestress is applied during erection while the resin is still fluid, within 15-20 minutes of application. This prestress creates a uniform compression of 200 lbf/sq.in. over the cross-section.

As the ribs are inclined at various elevations, the angle of the jointing face to the vertical ranges from 15° to 85°. During the erection the precast segments are supported on a travelling steel arch, which is removed as each rib becomes self-supporting. Subsequently the stresses across the glue line are complex and are caused by the applied prestress, the moments, shears and tensions.

The maximum and minimum compression likely under extreme conditions across the joint are 2500 lbf/sq. in. and -300 lbf/sq. in. respectively. The maximum shear stress is of the order of 500 lbf/sq. in. Under normal conditions the compressive stress is 1500 lbf/sq. in. and the shear stress is 100 lbf/sq. in. The segments were cast, stored and erected in the open with no weather protection.

The thin epoxy joints enabled

(a)
the geometry of the curve of the rib to be accurately set out in the casting yard and duplicated during erection in the air.

(b)
the ribs to be erected quickly. It is estimated that the time required to make a joint and leave it to cure before

	63/27W	63/27S
Mixing Proportions by wt.	100/22	100/17
Compressive strength, 16 hours at 25°C (lbf/sq. in.)	7300	5950
Compressive strength, 7 days at 25°C (lbf/sq. in.)	10,500	10,600
Tensile shear strength, 16 hours at 25°C (lbf/sq. in.)	950	680
Tensile shear strength, 7 days at 25°C (lbf/sq. in.)	1850	1850
Pot life 2-pint kit:- 5°C 25°C 32°C	2 hrs. 45 mins. 15 mins.	3 hrs. 1½ hrs. 30 mins.
Heat distortion point	71°C	68°C
Reduction in compressive strength after 2 hrs. at 250°C	4%	4.5%
Viscosity of mix at 25°C (Brookfield no. 4. at 20 r.p.m.)	19,000 ±1500 cps.	30,000 ±1500 cps.

full prestressing was about one-fifth of what would be required with cement mortar joints.

The epoxy resin system

The resin used was a polyamide-cured epoxy resin supplied by CIBA Company Pty Ltd. of Sydney. Two grades of hardener were used, 63/27W for winter and 63/27S for summer. The former contains an amine adduct as accelerator, the latter has an aminophenol accelerator. The resin, *Araldite* 63/27, contains *Araldite My* 752 (a liquid epoxy resin containing a reactive diluent), 200 mesh quartz flour, *Aerosil* and grey colouring paste. The properties of the two systems are given in the table on the left.

Originally an amine cured system was offered at the tender stage. It met the specifications for strength and had a higher heat distortion temperature. However, the better application properties (easier to handle, less sensitive to dampness) of a polyamide were considered to be more advantageous.

Surface Preparation

The concrete surfaces to be glued had been cast against each other to achieve matching faces. A bondbreak material was used to prevent adhesion during casting, and this had to be removed before glueing. To check the efficiency of various bondbreak compounds, and to assess their effect on the bond strength of the epoxy resin, a test programme was commissioned from the School of Civil Engineering, University of New South Wales, Sydney. The method used was to glue end-matched half beams to make 28 in. long beams for testing in flexure. After casting various surface preparation treatments were tried.

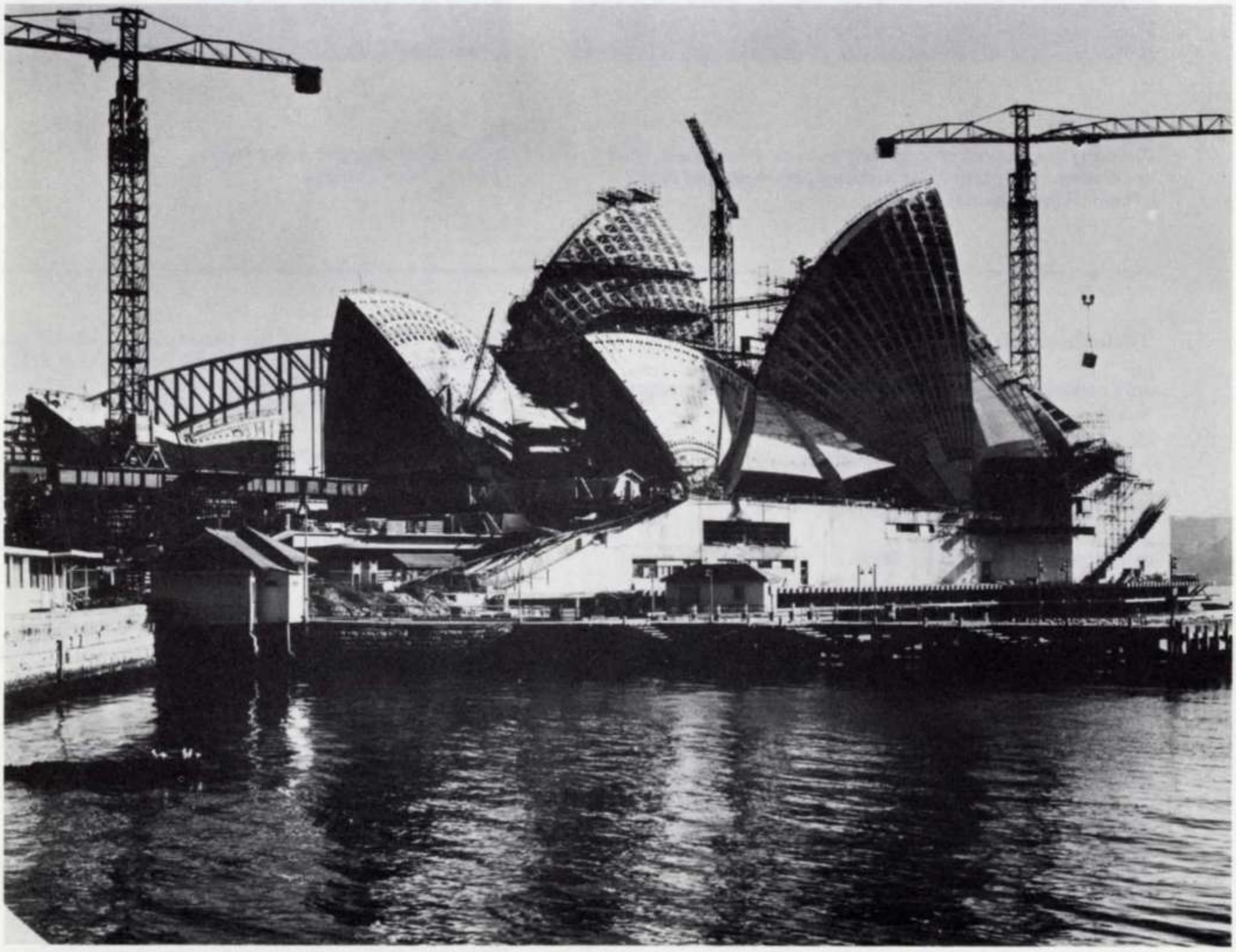


Fig. 14
General view of the Sydney Opera House under construction. (Photo: Max Dupain)

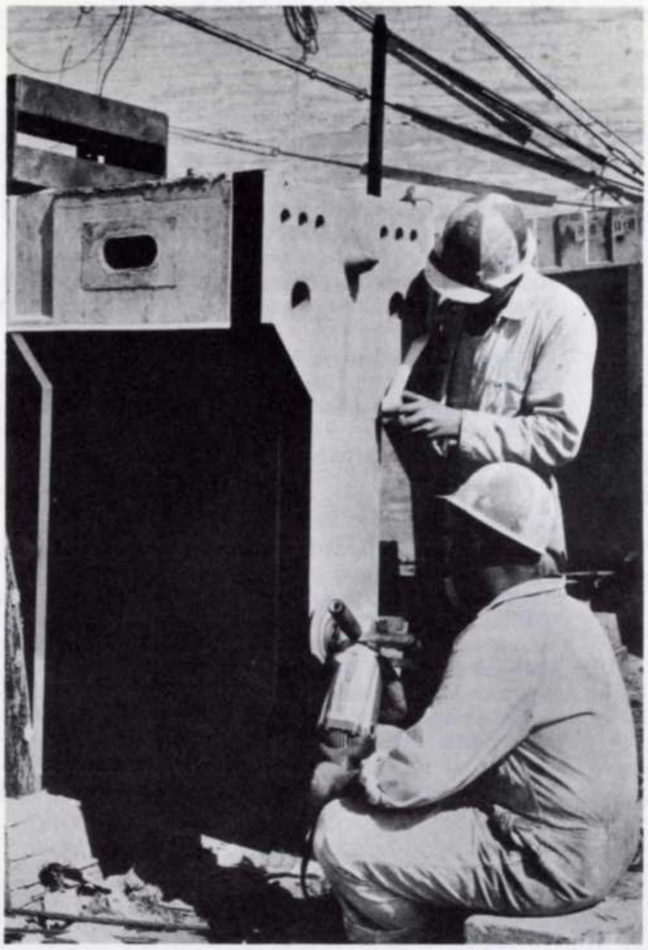


Fig. 15
Grinding the surface of a precast concrete rib segment prior to erection, and placing the masking polythene and tape. (Photo: Max Dupain)

Fig. 16
Lifting a rib segment into position. (Photo: Max Dupain)

These included:

(a)
acid etching with hydrochloric acid, followed by water washing.

(b)
a detergent scrub and water washing.

(c)
wire-brushing and water washing.

(d)
grinding away at 1/32 in. layer.

All the commercial bondbreaks and surface treatments were satisfactory in the laboratory, showing little reduction in strength. Field trials were carried out on full-size units using method (a) above. These failed badly, with bond failure over large areas. A stronger acid concentration seemed necessary.

The final procedure adopted which proved most satisfactory on site using a hydrocarbon resin as bondbreak was as follows: Grind off 1/32 in. of concrete not more than ten days prior to jointing. If this time is exceeded, regrinding is required. Wash down with absolute alcohol immediately before jointing to remove dampness and grease. No resin is applied to damp surfaces, flame drying being used if necessary.

Application techniques

Early laboratory trials showed little difference in strength between priming or coating both surfaces or only one. However, later tests showed that application to both surfaces was marginally preferable. The quantity of resin required for each joint was calculated and a small percentage extra allowed for squeeze out.

The epoxy was applied by trowel to the lower surface, spread so that there was some additional thickness at the corners, worked around the spigots and ducts (it was prevented from entering duct and bolt holes by means of a cork gasket glued to the lower surface). A roller was used to apply a thin layer of the same epoxy to the upper surface.

Adjacent surfaces were protected by a polythene skirt fixed by masking tape which extended $\frac{1}{4}$ in. onto the jointing surface. All segments were treated with a light coating of a p.v.a. emulsion prior to erection to prevent adhesion of spillage from above. This emulsion weathered off in time.

Shimmed Joints

With matching faces it is sometimes necessary to accommodate angular changes at the joint to maintain correct alignment. This may be done by forming a tapered joint using the following technique:

(a)
Steel shimms are used to control the direction of the rotation (the area and position of the shimms would be such as to prevent spalling or local crushing of concrete).

(b)
$\frac{1}{8}$ in. steel spacers are glued to the surface whenever an epoxy thickness of more than $\frac{1}{8}$ in. occurs. The shimms and spacers should cover about 60% of the area and the effective epoxy thickness would not exceed $\frac{1}{8}$ in.

(c)
The calculated amount of epoxy required would have an allowance of 50% for squeeze out.

(d)
The application procedure would be otherwise similar to that of standard joints.

This technique was used whenever necessary. The restricting of the glue-line thickness was considered necessary to limit the total amount of creep.

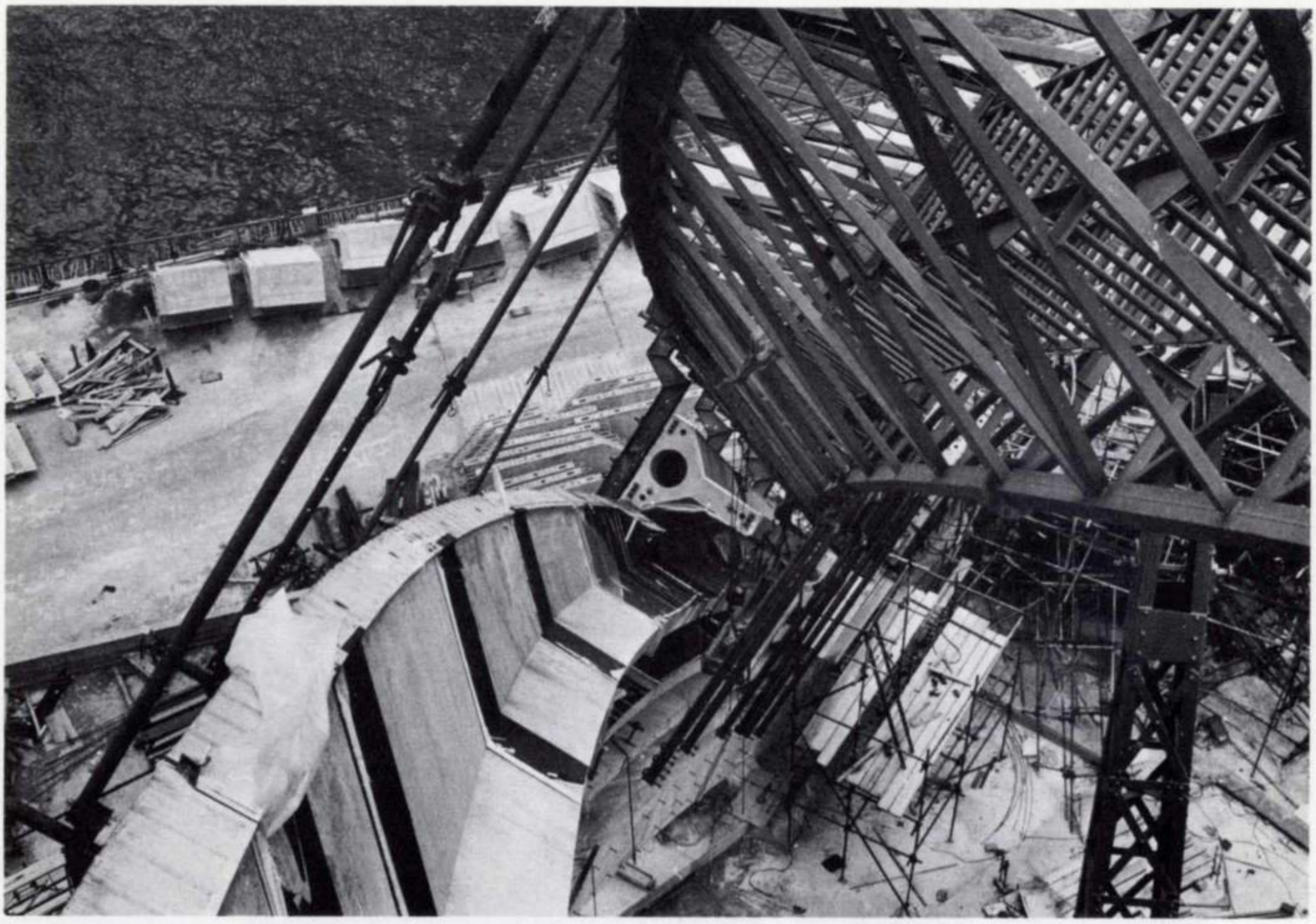


Fig. 17
View down a rib under construction.
(Photo: G. Wood)

Scabbled Joints

One set of joints were inclined at 70° to the axis of the ribs, and a shear stress due to the prestressing equal to 0.35 of the longitudinal stress existed. Owing to the danger of cracking of the epoxy joint during erection, and because the epoxy/concrete coefficient of friction in attached joints is about 0.35, the joint was lightly scabbled. The surface preparation was to clean the scabbled surface with a wire brush to remove any loose pieces of aggregate, dust off with a soft brush and clean with alcohol in the usual way.

About 1 sq. ft. of unscabbled area was left at each joint and across this a prestress of only 30 lbf/sq. in. during jointing was applied. On some of these joints insufficient resin was trowelled on and some downward flow occurred. After stressing a gap was left in the joint. The repair technique was to create shear keys in the joint by drilling six 1 in. holes at 2 in. centres, filling with epoxy and forcing 1 in. steel rods into the holes, thus squeezing the fluid epoxy into the surrounding joint. Grouting with a special epoxy was also carried out to completely fill the joint. The shear keys were used as the bond strengths of epoxy to fully cured epoxy is not good.

Other erection problems

At an early stage in the job, trouble was experienced with the jointing of some segments of one of the main arches. Insufficient prestress was applied across the fluid epoxy joint to hold the precast segment in position and angular rotation took place at the joint. This created internal voids within the epoxy layer covering up to 50% of the total area. This was not noticeable from the outside. However, demounting revealed large unbonded areas of resin/concrete interface.

In addition, movements of the scaffold lead to cracking of some joints. These were repaired by an injection technique:

(a)
drill holes into joint at 12 in. centres

(b)
seal the full perimeter length with epoxy resin putty

(c)
protect surrounding concrete against leaks and bursts

(d)
inject epoxy resin grout from the bottom.

Control testing on site

In addition to their use for rib segment jointing, epoxy resin formulations have been used for patching mortars, ceramic tile adhesive, sealant primer, drill hole fillers, waterproof membranes and joints. It was therefore necessary to have careful control to ensure that the correct mix was used for each job.

This control was achieved by carrying out exotherm and density tests on each batch of material. The measurements of peak exotherm temperature and time were carried out in accordance with the Society of Plastics Industries Specification ERF 2-61 method C. The temperature for each test was $70^\circ \pm 2^\circ$F. The density tests were carried out using BS 2782 Part 5: 1965, Method 509A. The samples were demoulded 4 hours after they had gelled, and were cured for 48 hours at $70^\circ \pm 2^\circ$F.

Limits were specified for each resin formulation giving the range in which the values must fall. In the case of the exotherm test the maximum time and minimum temperature were the important limits. The acceptance limit for density was $\pm 5\%$ of the nominal value. If either test failed, flexure and Shore hardness tests were carried out.

It is believed that these controls enable the following errors to be detected; labelling error, wrong components, omission of components (e.g. accelerators) and the presence of moisture in fillers.

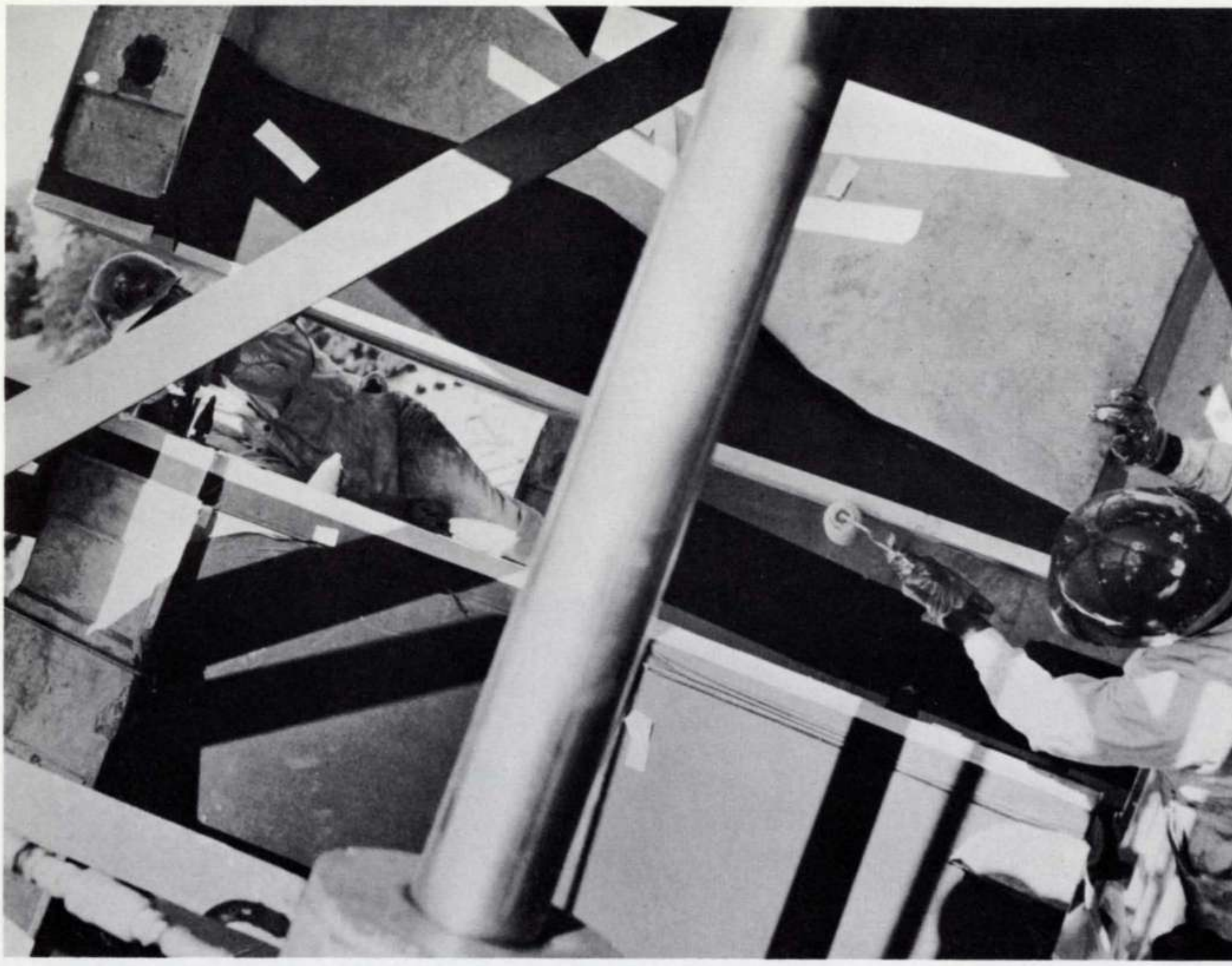


Fig. 18 left
Holding a rib segment in place while glueing is carried out. (Photo: Max Dupain)

Fig. 19 above
Applying the epoxy resin adhesive to both surfaces. (Photo: Max Dupain)

GENERAL COMMENTS ON RESIN ADHESIVES

The experiences of the use of resin adhesives on the jobs described above have led to the conclusion that they have very definite advantages for jointing precast concrete units. In all cases the techniques were successful and none of the job engineers responsible for those structures has said that he would have made the joints any different another time.

However, certain comments must be made about the use characteristics of the materials.

1.
Bad joints can look good and so they are difficult to detect. Unless the joint can be tested for strength it is essential to set up a procedure technique and site supervision such that bad joints are unlikely to occur. There is a need for non-destructive testing methods that can be applied in the field. There is no fail safe with epoxies in general. Either bond is achieved or it is not. Because poor joints may look satisfactory, a degree of complacency can develop which makes detection difficult.

2.
It is difficult to keep surfaces clean when mechanical lifting equipment is used. Handling of the surface can occur unnoticed. The effect of a small amount of grease from handling is not known.

3.
The degree of care required for successful use of epoxy formulations is higher than that of standard building practices. Specially trained personnel are required for the wide use of resins on construction sites (for jointing, patching, sealing etc.). There is considerable danger that malpractices will be undetected.

4.
Rapid changes of ambient temperature may cause difficulty in selecting the right grades of adhesive.

5.
Little is known of the effect of weathering of a prepared concrete surface on the adhesion of a resin. At Sydney the evidence suggested that a limit of 10 days should be put on the interval between grinding and glueing. This appeared to work satisfactorily.

6.
Account must be taken of the fact that joints made between precast concrete units with resin adhesives are more rigid than cement mortar joints. Attempts to readjust alignments after the resin has gelled can lead to cracking of units, instead of minor cracking of mortar.

It is quite clear that considerable research needs still to be done on the various topics outlined in the introduction. Only with more detailed information will designers consider the wider use of structural adhesives. In addition, the various authorities who issue standard specifications for materials, must start now to draw up ones for these materials. This will avoid any dangers of designers being sold 'structural adhesives' with very inferior properties.

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