THE ARUP JOURNAL

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Contents

The cover, front and back,
shows folded plates,
especially drawn by Poul Beckmann

Folded plate roofs*

David Dowrick

*Editorial note:
D.V., this article will be
defended by David Dowrick at the second
Technical Staff Meeting on 8 February.
Everyone is welcome to attend
and contribute to the discussion.

GENERAL
What is a folded plate? If we answer this quite truthfully
the answer is almost any structure apart from a column
that we care to name.

FOLDED PLATE RECIPE
Minimum ingredients
1. Two flat structural elements (capable of taking shear in
    their own plane).
2. One common boundary between them.
3. One rigid support.
Making Procedure
1. Place the two elements together, but not in the same
    plane.
2. Join the two elements firmly together along the common
    boundary.
3. Attach both parts of the resulting 3D structure firmly
    to the support.
4. Allow to stand, and observe.
If what you are looking at is similar to figure 1(a) you now
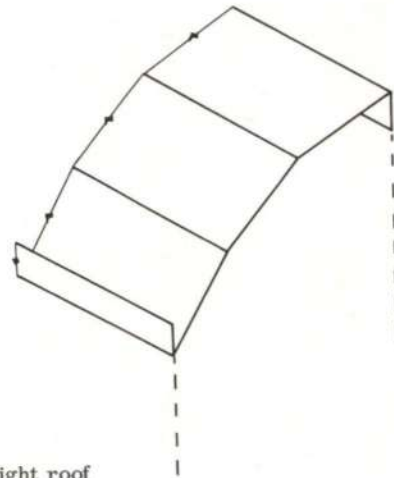have a simple folded plate structure.
From this simple beginning a great variety of structures
can be built up including silos, north-light and non-prismatic
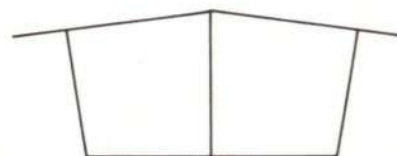roofs, box beams for bridges (figs. 1b - 1e). By way of

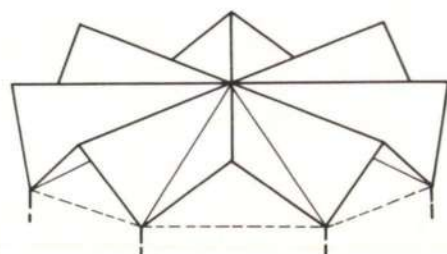Figs. 1a-1e Right
Some folded plate structures
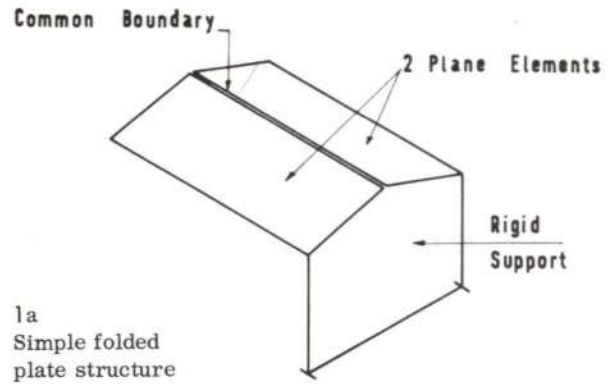
1a
Simple folded
plate structure
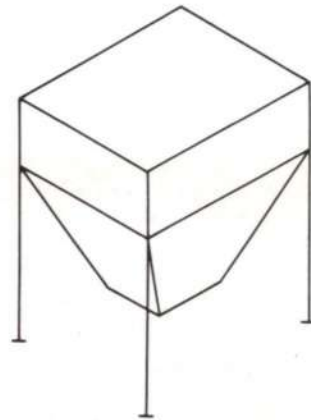
1b
Hopper

1c
North light roof

1d
Box beam

1e
Non prismatic roof

Figs. 2a-b
Folded plates v shells

definition, a prismatic roof is one composed of rectangular plates joined together, such that each plate is parallel to a common longitudinal axis. The variations of folded plate structures are numerous and this discussion will be limited to roofs.

Because of the simple physical concept and the almost unlimited geometric configurations, this medium can be very popular with architects. The inherent plastic possibilities and the creation of various types of internal and external space are great attractions to them and as the roofs can be made to span long distances without columns, free planning is facilitated. Added to the spatial freedoms are those of surface texture, as the structure can be made from various materials, concrete, steel or timber.

The engineer also enjoys the use of folded plates, though not all of the reasons are altruistic. On occasions he is able to span large spaces. The spaces and analysis may be intriguing and in some offices folded plates will be a change from routine. It may also involve him in an unfamiliar material and new detailing techniques.

As for the client he also may have reasons for satisfaction.
1. Economy. Folded plates tend to be efficient structurally and simple to fabricate (plane surfaces) though neither is necessarily true.
2. Functional suitability. He may very well get his share of what Ove Arup calls 'commodity', whether designed by Arups or not!
3. Visual and environmental pleasure. Such a roof may provide relief from monotony or even positive excitement.

On the negative side of the argument, folded plates can be treated in a gimmicky fashion, by any of the perpetrators, be they architect, engineer or client. Let us not use them too facilely. Economy and commodity should always be satisfied. Unfortunately they won't always.

Folded plate roofs are popular for industrial buildings, churches, schools, exhibition halls and various places of assembly, for all of which they are well suited. The modern theory is only about 40 years old, started by German engineers particularly Craemer [2], but for some rather obscure reasons they seem more popular in the New World than elsewhere. They are widely used in the United States and various Commonwealth countries, especially in Africa. In the United States labour costs are high, in the other countries mainly low, so the reason for the New World partiality to folded plates is not a simple economic difference from Europe.

We have broad experience in folded plate roofs, principally due to our African practice. If I may dare to mention a competitor, Felix Samuely seems to have done quite a lot in Britain. Nervi sometimes does them to perfection.

STRUCTURAL ANALYSIS
Fear not! I won't trouble you with detail. One or two not very inspiring textbooks exist, of which Born's [1] is worth looking at, together with a welter of post-war mathematical argument in technical magazines, mainly American. Most study the classical concept of the load being resolved into the planes of the plates which span mainly in their own plane, interacting with their neighbours with shears along their common boundary - see recipe. The load component normal to the plates is carried in lateral bending spanning between folds. As most techniques involve solving a set of simultaneous equations, the computer comes timely to our aid.

Amongst the published papers on folded plate analysis are some which advocate simple beam analogy but these should be treated with some suspicion. The best place to look for a recommendation on design methods is the American Society of Civil Engineers' report on folded plate construction [3] published in December 1963.

Folded Plate

Shell Vault

Fig.2a

Folded Plate

Simulation in Shells

Fig.2b

There is also an interesting paper on reinforcement of reinforced concrete plates by Guralnick [5].

Shell roofs and folded plate structures are closely related, just as a circle can be considered as the limit case of a series of short straight lines (fig.2a). For prismatic structures, attempts have been made to treat shells as folded plates and vice versa. Gibson [4] at Manchester has developed very promising computerized solutions for various folded plate vaults treated as a series of large radius interconnected shells (fig. 2b).

CONSTRUCTION IN CONCRETE
Perhaps the most adaptable material for folded plates is concrete and a considerable range of constructional techniques is available whether in situ or precast, prestressed or plain reinforced concrete. Spans ranging from 30 to 120 ft. are commonly employed, while the largest known to me is nearly 190 ft.

IN SITU OR PRECAST
In many cases the final appearance of the roof will not differ much whether in situ or precast, so the choice will often be influenced more by economics, the type of site and contract, and the preferences of the engineer and contractor, rather than by the architect and his concern for surface finish or articulation. Precasting is popular in the United States, which no doubt results from their high degree of mechanization, especially cranage, and a constant dollar-inspired desire to reduce time on site.

Prismatic roofs are generally precast in complete bays, each bay being an isolated folded plate for its dead load (fig.3) although complex roofs are sometimes made from plane elements stitched together (fig.14c).

PRESTRESSING
Both post- and pre-tensioning are used. One American firm was selling so many standard roof units that they were able to set up a long-line pre-tensioning bed (fig.4). An example of the finished products is shown in figure 3. [8]

Prestressing, as with other structures, assists in long spans. It is used with both precast and in situ work, though the latter case is less popular because of inherent difficulties in transfer of load to the supports, in allowing for movements of the roof due to shortening under prestress and in correct distribution of prestress throughout the structure. Perhaps the most ambitious example of a folded plate roof is that for the new
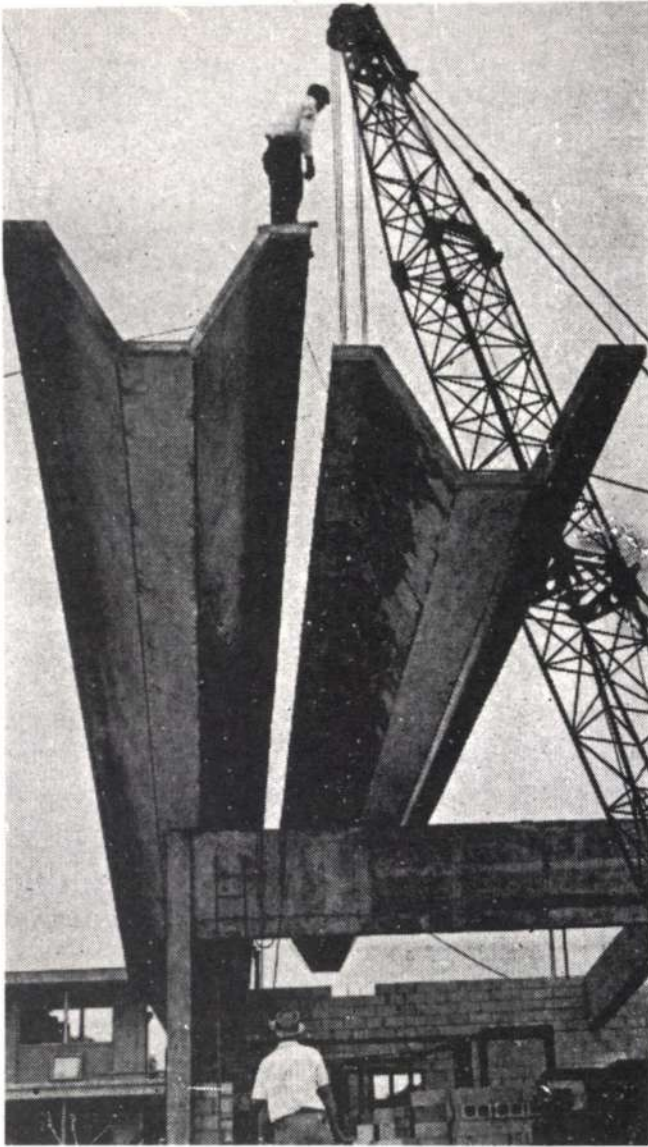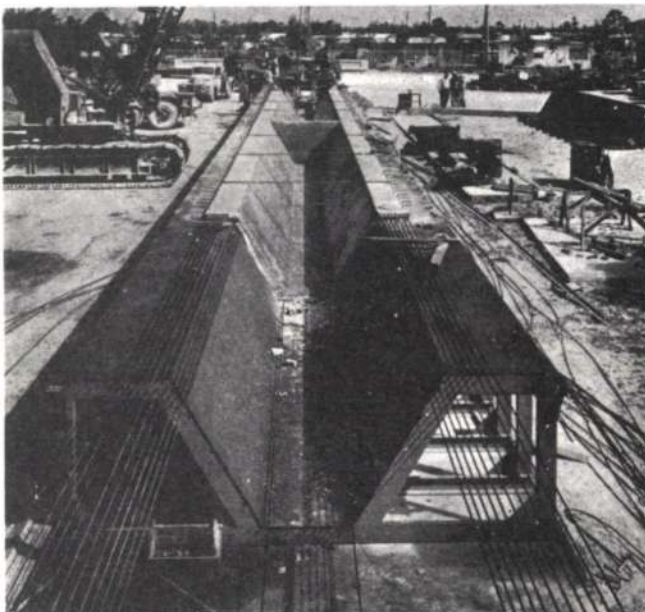
Fig.3 above
Precast production line units, Florida
(reproduced with permission from
American Concrete Institute Journal)

Fig.4 below
Long line pretensioning of roof units, Florida
(reproduced with permission from
American Concrete Institute Journal)

town hall in Marl in the Ruhr, West Germany (figs. 5 to 8).
This is in situ, post-tensioned and spans almost 190 ft. The
building is built to take mining subsidence and its portal
frame has pins and a sliding support making it statically
determinate. Apart from the advantage this has in earth
movements, it also avoids complications due to movement
under prestress but it is a pity that the sequence of
stressing was not described in the technical paper [7] as
this poses interesting problems. The whole roof, comprising
1400 cubic yards of concrete, was poured continuously in an
80 hour period, a quarter of the prestress was applied two
days later, presumably to control shrinkage cracking.
This building, while ambitious in scale, seems to use far
thicker plates in the roof than necessary.
Another post-tensioned structure is shown in figure 10.

WATERPROOFING AND DRAINAGE
Folded plate roofs, almost by definition, provide natural
valleys to guide water to the supports and concrete ones
sometimes need little other than a screed giving extra fall
in the valleys to give adequate watertightness (fig. 9b). As
the concrete mixes tend to be stiff and therefore fairly
dense they are naturally reasonably waterproof but only if
low tensile stresses exist in the concrete. In Africa, we
quite frequently use 1/2in. cover to top steel in in situ roof
slabs poured on a slope without any top shutter, e.g.
Salisbury Market Hall. This type of roof is then generally
waterproofed with bituminous felt. Feldman's Warehouse
in Vereeniging was given a top membrane of waterproof
paint. Occasionally no waterproofing membrane is used,
but more than 1/2in. top cover would obviously be called
for.
In concrete construction the roof water is typically led
away in downpipes cast into the columns (fig. 9a). It is as
well to provide overspills in case the downpipe entrance
becomes clogged. Sometimes the geometry of the roof is
such that water could fill up to the ridges should the drains
become blocked. This loading should be allowed for in the
design at a suitably reduced load factor.
There are, of course, various sealants which may be applied
to concrete roofs, some not very good. A good modern
treatment consists of a layer of neoprene covered by a
finishing coat of HYPALON. Of this type of roof coating,
generally termed elastomeric, a trade name to remember
is CASCO-DUR. It seems durable and comes in various

Fig.5
Town Hall, Marl, W.Germany. Top shutter
(reproduced with permission from
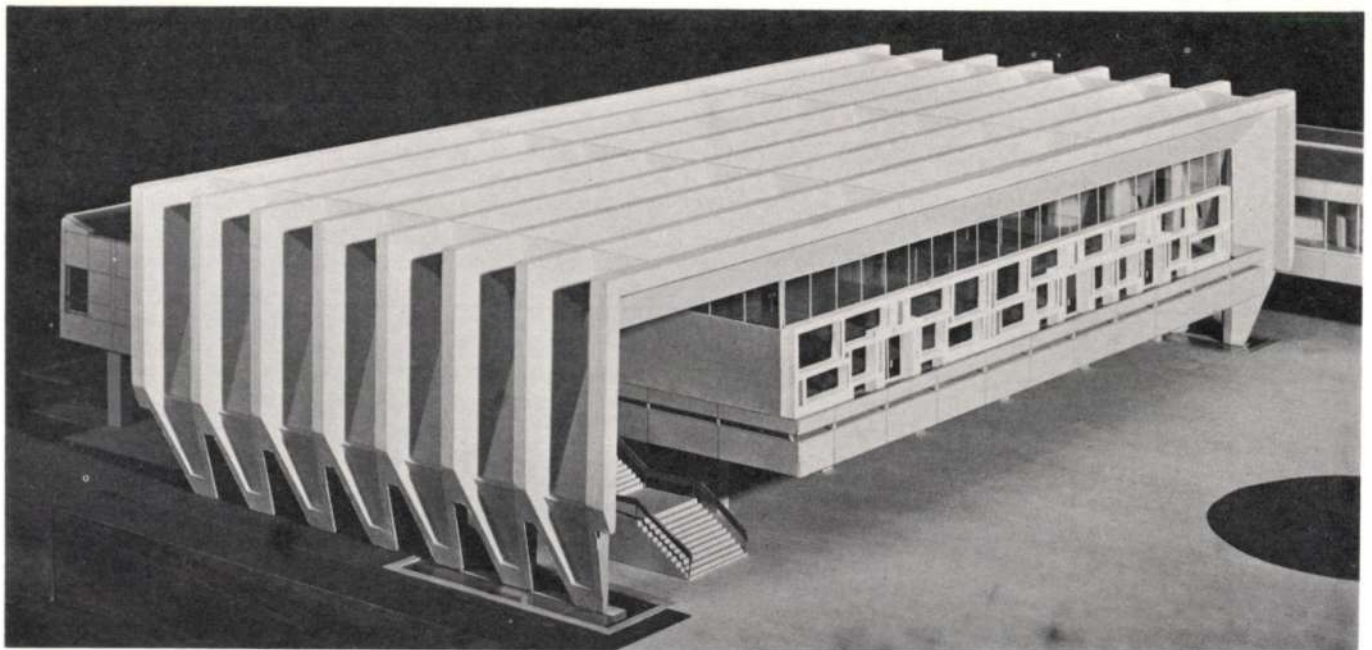Beton-und-Stahlbetonbau)

4

Fig.6
Town Hall, Marl, W.Germany
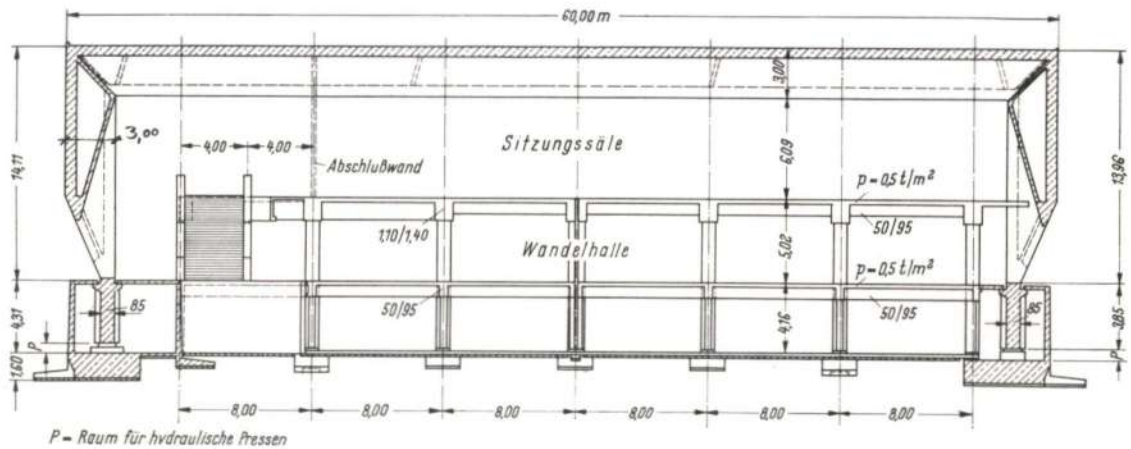(reproduced with permission from Beton-und-Stahlbetonbau)

Fig.7
Town Hall, Marl, W.Germany. Longitudinal section dimensions in metres
(reproduced with permission from Beton-und-Stahlbetonbau)

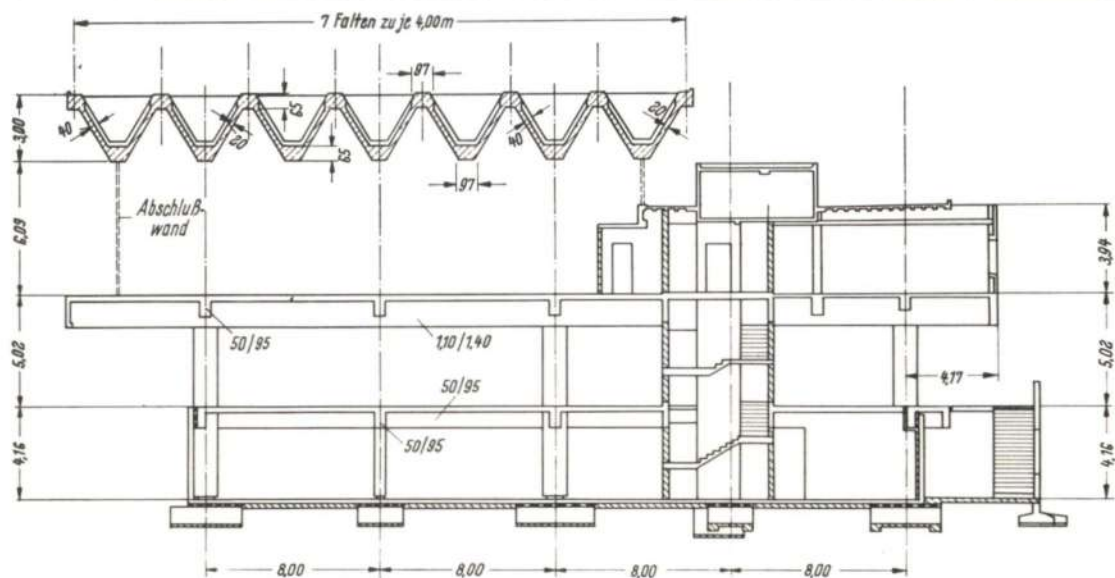Fig.8
Town Hall, Marl, W.Germany. Cross section, dimensions in metres
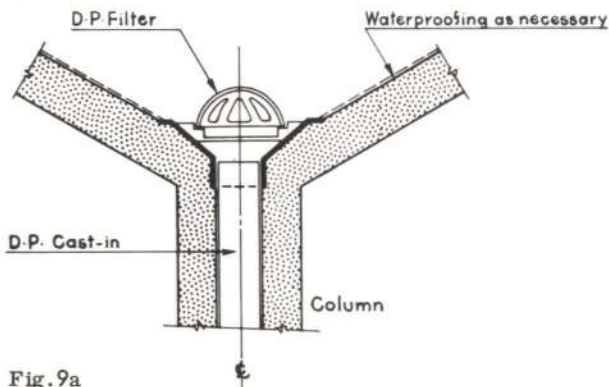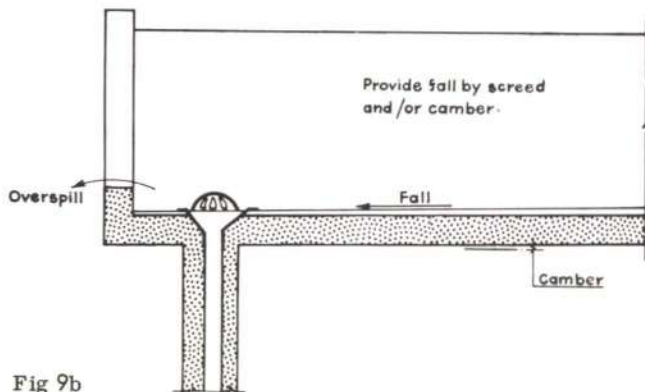(reproduced with permission from Beton-und-Stahlbetonbau)

Fig. 9a

Fig 9b

Figs. 9a-b Drainage details

permanent colours including white, the latter being a help for insulation in hot weather. As might be expected it is necessary to pay for quality. CASCO-DUR costs about 24/- per square yard compared with 15/- for reinforced bitumen.

INSULATION

On large roofs and especially in hot climates, it is good practice to minimize temperature effects by insulating the top of the slab. A vermiculite or similar screed is often used for this purpose. On Etco House, Nairobi, we covered the concrete with corrugated asbestos sheeting, which combined waterproofing with insulating.

Fig. 10
57 ft. span post-stressed roof, Indiana State College (reproduced with permission from Engineering News Record)

CONCRETE PLACING

Unless you are Felix Candela, placing concrete on slopes greater than about 40° is tedious without a top shutter, though up to this limit concrete of normal low slump (around 1 in.) is manageable but some care has to be taken in the concrete mix design. Figure 10 shows a slope of about 45° being concreted without a top shutter, the concrete mix being obviously very stiff. Figure 11 shows a contractor opting for top shutters for a similar slope.

Where speed of concrete pouring is essential, such as in a long-line factory, top forms are no doubt an advantage as employed in figure 4. Top forms are shown in position in figure 5 where the slope is very steep, 60° to the horizontal.

The thickness of concrete plates varies considerably, depending largely on geometry. For medium spans of 30-60 ft. the plates are generally 3 in. to 4 in. thick. Greater spans will not necessarily require thicker webs, as this is largely a function of the lateral span, e.g. in figures 6, 13a, the unit can span 120 ft. and is only $3\frac{1}{2}$ in. thick but the bay width is only 8 ft. At the other end of the scale the Town Hall at Marl is unbelievably heavy with webs 7 in. thick at mid-span, 14 in. at the support.

Concrete strength is determined by span and naturally increases for prestressed members. 3,000 - 4,000 psi is common for medium span reinforced concrete roofs. Daywork construction joints in a multibay roof are most conveniently made along a horizontal line just above a valley (fig. 12a).

CONCRETE DETAILING

Normal good practice suffices for most reinforced concrete plates but with thin plates special attention must be given to ensuring that correct cover is obtained, i.e. that top steel is accurately placed. Roof-lights require special attention to the reinforcing detailing, as holes are always stress raisers and require care in zones of high shear. Rounded rather than square corners to such apertures are an advantage, though this can be expensive.

Figure 12a shows a typical in situ reinforcing section. In order to gain the benefit of folded plate action, tie forces must be provided at or near the supports to permit the lateral arching action. This may be done by a simple tie member in steel or concrete (fig. 12b), by buttressing or

Fig. 11
150 ft. span in situ roof, Minneapolis-St Paul air terminal (reproduced with permission from Engineering News Record.)
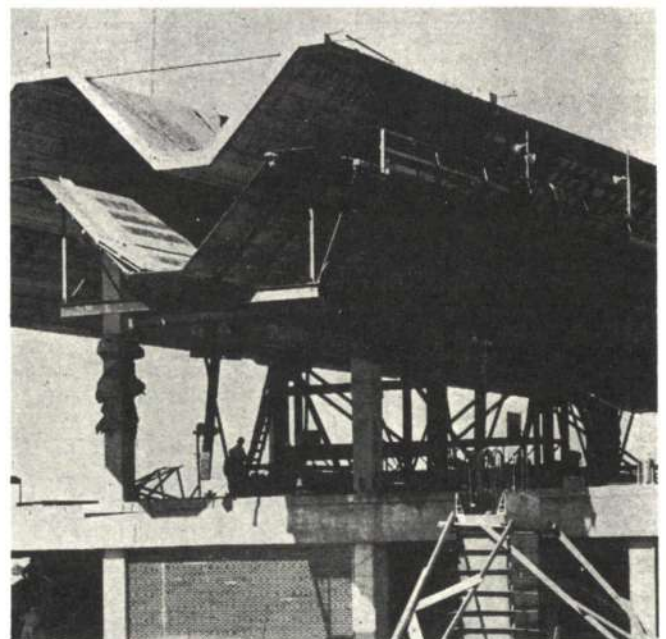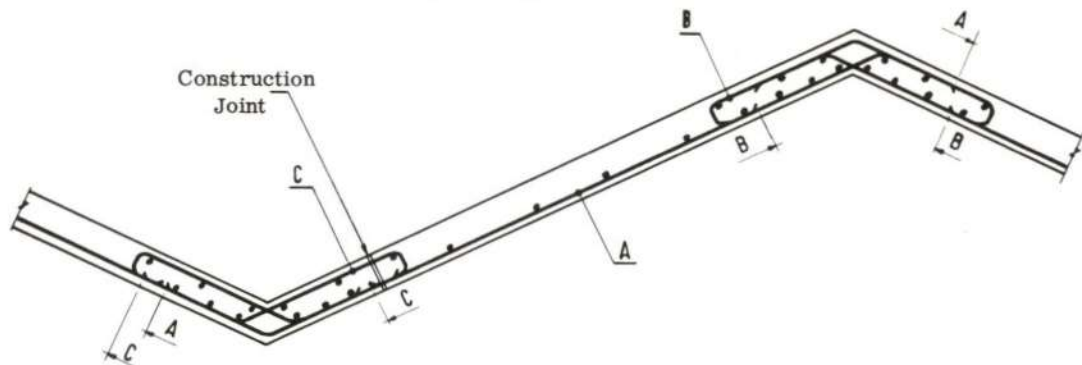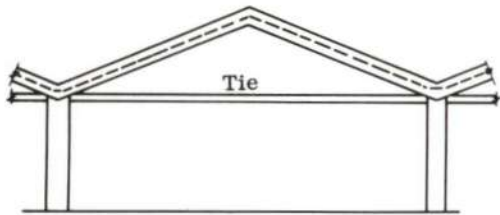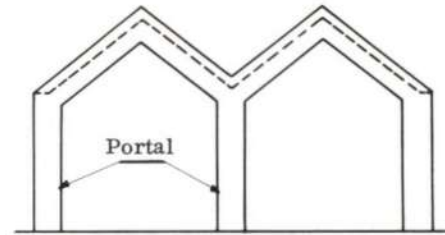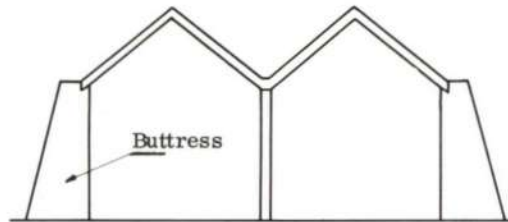
Fig.12a Typical reinforcement

Fig.12b

Fig.12c

Fig.12d
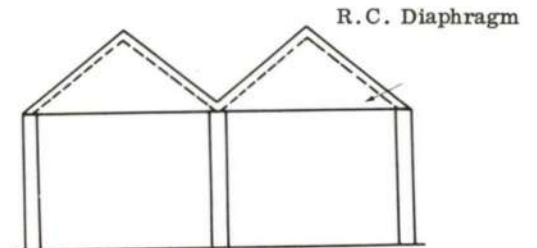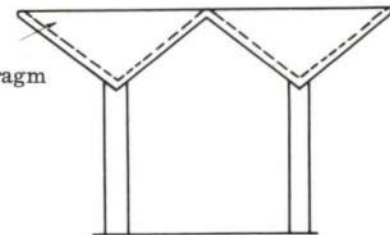
Fig.12e

Fig.12f

Figs.12a-f
Typical reinforced
concrete section
and supports

portal frames (figs. 12c, 12d) or by some form of diaphragm wall. The latter is sometimes below the roof line, also acting as part of the external wall (fig. 12e) or can be above the roof line (fig. 12f). In the latter case the diaphragm is sometimes sloped out of the vertical plane like an inverted hip roof, making a pleasant visual feature.

An interesting tie was provided by us for the Royal Technical College Library, Nairobi, in the form of a 3 in. outside diameter steel tube, the architect wishing to play down the presence of the tie member. The tube was also used to house 12/7 mm. prestressing wires which were stressed to counteract lateral spreading of the roof at de-prop, as we were worried about the effects of such movement on some of the structure below.

Precast concrete induces its own peculiar detailing problems related to jointing. Figure 13a shows a pre-tensioned unit in section. Note the ties provided to complete the structure during handling and erection. These could be dispensed with later as long as a suitable connection was made between adjacent precast members. A connection detail which is little more than for weatherproofing is shown in figure 13b. It does leave the joint boldly expressed from the underside while being constructed entirely from the top. Figure 13c

shows structural continuity achieved by a comprehensive in situ stitch. This type of jointing is often applied to non-prismatic roof shapes (fig. 1e). Welding of the reinforcement as shown may be necessary in thin plates where a mechanical reinforcing joint would be too bulky.

STRESSES

As in shells, if the plates are very thin compared with their width, the compressive working stresses should be kept low to about 1/6 of the cube strength. If the plates are not prestressed and are not to be given a waterproofing membrane, the tensile working stresses should also be kept low. The limit could be about 1/12 to 1/15 of the cube strength, depending on the degree of waterproofness required and the configuration and detailing of the roof with respect to water resistance and run-off.

In other circumstances normal reinforced concrete working stresses can be allowed.

Permissible stresses may dictate the geometry of the roof, as apart from changing with the span size the stresses may be modified by changing the pitch of the roof slopes. A change in slab thickness generally only modifies the lateral stresses, longitudinal stresses remaining constant.

VARIOUS FOLDED PLATE ROOFS

No.	Building	Material	Method	Span	Section
1	Town Hall, Marl, West Germany.	P.S.C.	In situ	187 ft. c/c of support walls	13'-3" t = 8" mid-span 60° 9'-10" 7 Bays @ 13'-3"
2	Exhibition Hall, Civic Centre, Seattle.	P.S.C.	In situ One bay stressed at a time One set of forms in four pieces on scaffolding	140 ft.	20'-0" 3'-6" 3'-6" 6'-4½" 47° t = 4½" Constr. Joints 11 Bays @ 20'-0"
3	Avocado School, Homestead, Florida.	R.C.	Precast Production line	≤ 60 ft.	8'-8" t = 3" 2'-0" 60° 1'-4" 1'-6" 3'-0"
4	Various Buildings from factory in Florida.	P.S.C.	Precast	≤120 ft.	8'-0" t = 3½" 3'-9" 60° 1'-0" 2'-0" 2'-0"
5	Library, Central College of Education, Washington.	P.S.C.	Precast	65 ft.	
6	Airport Terminal Minneapolis – St Paul.	R.C.	In situ	150 ft.	30'-0"
7	Kiln Building Castle Hayne North Carolina	R.C.	Precast	90 ft.	100 tons 50 tons 25'-0" (approx.) 9'-0" (approx.)
8	Gymnasium Indiana State College Terre Haute	P.S.C.	In situ	157.5 ft. Longest in USA Aug. 63.	26'-0" 11'-6"
9	Warehouse Colorado	Timber	Prefabricated on site	67 ft.	50'-0" ¾" plywood on top 8"x2" joists @ 2'-0" c/c

Span Depth	Top Shutter	Concrete	Construction Notes	Drainage and Waterproofing	Reference
19	Yes	1400 cu.yds conc. poured continuously in 80 hours from one side to the other. Half the tendons were half stressed 2 days after concreting.	Portal on roller bearings for mining subsidence; this also useful for eliminating secondary effects when stressing. 3200 kips prestress per roof Vee.	Drainage down end columns. Roof diaphragms perforated.	F. Vaessen (7)
22	-	2 in. max slump $\frac{3}{4}$ in. max. aggregate 4500 psi cylinder strength at 3 days. One bay per week.	2 - $1\frac{3}{4}$ in. dia. tie rods hidden in masonry wall at ends. 6 $\frac{1}{2}$ in. dia. strands in 5 $1\frac{3}{4}$ in. dia. ducts in each web i.e. 1200 kips prestress/bay.	2 in.dia. downpipes in columns which were 40 ft high and precast	A.C.I. Jnl. v.60. No.6 Part I June 1963 pp. 705-9
21	Yes	3750 psi cyl. str. 24 hour turn round on production line.	Designed for 120 m.p.h. hurricane, at working stress. Load factor of 2 gave net uplift of 26 psf. Galvanized tie rods at supports. Cambered forms. Galvanized mesh reinforcement.	See valley detail in text (Fig.13b). Joint detail to hide differential deflections, and take up tolerance. Camber provides runoff	W.C. Harry (8)
32	Yes	6000 psi cyl. str. at 28 days. Stressed at 48 hrs.	Tie rods at 10 ft. crs reduced lateral reinforcement. Cost in place $2.50 per sq ft. The units weighed 60 psf compared with 80 psf for equivalent Tee beams.		(8)
-	Yes	Steam curing and high early strength cement.	98 units cast from 2 moulds on site. One unit per day per mould.		Eng. News-Record (9)
-	Yes		Plywood forms, on steel frames mounted on rails. 17 bays cast in place with one set of forms.		(9)
-	Yes	Lightweight concrete Steam cured	P.C. units lifted by two mobile cranes 65 ft up to roof level. Then winched along on rails; when in position jacked up and finally lowered on to neoprene pads.		(9)
-	No		8 bays from one bay of forms on 30 ft high scaffold. Forms pinned to concrete, and lowered after shifting scaffold to next bay.		(9)
14	N.A.	N.A.	50 ft wide Vee section prefabricated, erected with mobile crane. 3/4 in. plywood on top 5/8 in. plaster board fixed to under side after erection.		(9)

SOME ARUP FOLDED PLATE ROOFS

No.	Building	Material	Method	Span	Section
10	David Greig Shop, Canterbury, G.B.	R.C.	In situ	39 ft.	11'-6"; 2'-0"; t=3"; 30°; 3'-3" (approx). 5 Bays @ 11'-6"
11	Castle Brewery, Salisbury, Rhodesia.	R.C.	In situ	2 spans each 45 ft. 6 in.	1'-9"; t=4"; Glazing; 35°; 9"; 1'-3"; 7'-6"; 4'-0"; 10'-6"; 3'-6"
12	Market Hall, Salisbury, Rhodesia.	R.C.	In situ	2 spans 88 ft. and 20 ft.	t=4"; 4'-3"; 2'-6"; 30°; 5'-9" Rise; 3'-0"; 20'-0". 6 Bays @ 20'-0"
13	Library, Technical College, Nairobi.	R.C.	In situ	2 spans 42 ft. and 51 ft.	t=4"; 25°; 5'-3" Rise; 2'-6"; 22'-6". 8 Bays @ 22'-6"
14	Stadium Canopy, University, Kumasi, Ghana.	R.C.	In situ		44'-9"; tie beams; 1'-3"; 4'-6"; 23'-7"; Cols. @ 15'-6". t=3"; 5'-2"; 57° max. (varies). 17 Bays @ 5'-2"
15	Keartland Press, Heriot, South Africa.	R.C.	In situ	3 spans each 77 ft.	1'-6"; t=5"; 2'-4"; 18°; 5'-0" Rise; 1'-8"; 30'-0". 7 Bays @ 30'-0"
16	S.A.A.F. Memorial, Swartkop, South Africa.	R.C.	In situ		17'-0" (approx); 14'-9"; 47'-0". PLAN. Simplified Views; max. slope = 50°; Supports; 19'-0"; 11'-3". A-A; Supports
17	O.K. Bazaars Store, Orange Grove, South Africa	Steel	–	83 ft. (max.)	6" R.S.C's; 27°; 8'-0"; 31'-0". 5 Bays @ 31'-0"
18	Dothill Infant School, Shropshire, G.B.	Timber	Prefabricated		46'-0"; 40'-0"; HIP; Column; VALLEY; 10'-0"; 11'-3". PLAN. 3/8" plywood both sides; 4"x1½" joists @ 16" c/c; 15°; 3'-0". ELEVATION

| Span Depth	Top Shutter	Concrete	Construction Notes	Drainage and Waterproofing	Office & Date
12	No		End diaphragms above roof soffit.	3 in. downpipes in columns	London office Job 758 Built 1954
6	No	$1 : 1\frac{1}{2} : 3$ concrete	End diaphragm wall under roof soffit.	3 in. downpipes in columns Roofing felt	Johannesburg R32 1957
15	No	$1 : 1\frac{1}{2} : 3$ concrete	End diaphragms under soffit. Note provision of ridge lights. $\frac{1}{2}$ in. cover to reinf. top and btm.	External downpipes Roofing felt	Johannesburg R30 1957
9	No	3300 psi cube at 28 days	Structural ties of 3 in. o.d. m.s. tube, which was post-tensioned with 12/7 mm wires to protect lower works from roof spreading movement.	External downpipes	Nairobi 1451 1961
-	Yes	$1 : 2 : 4$ concrete $\frac{1}{2}$ in. max. aggregate	Tie beams cast first with key ways and starters to folded slab; which was then cast one 17 ft bay at a time. Lateral bars $\frac{1}{4}$ in. dia. $4\frac{1}{2}$ in. crs. Long l. bars $\frac{1}{2}$ in. dia.	Fall of 1 : 12 along valley, no downpipes water falling freely off back. No special waterproofing, 1 in. cover top and btm.	Accra G261 1963
10	No	3000 psi cube at 28 days	Prestressed tie beam at valley level.	4 in. dia. downpipes in columns.	Johannesburg SA665 1966
-	No	$1 : 2\frac{1}{2} : 3\frac{1}{2}$ concrete	Tie force created by buttress columns. The concreting would have been better if a top shutter had been used.	The roof was covered with terrazzo plaster (articulated by nylon strips).	Johannesburg SA412 1962
10	N.A.	N.A.	Shop welding, site bolts, Depropping of valleys carried out well behind bay being erected. M.S. tie at valley level.	Valley detail - see Fig. 14b in text.	Johannesburg SA165 1966
-	N.A.	N.A.			London 1285 1960
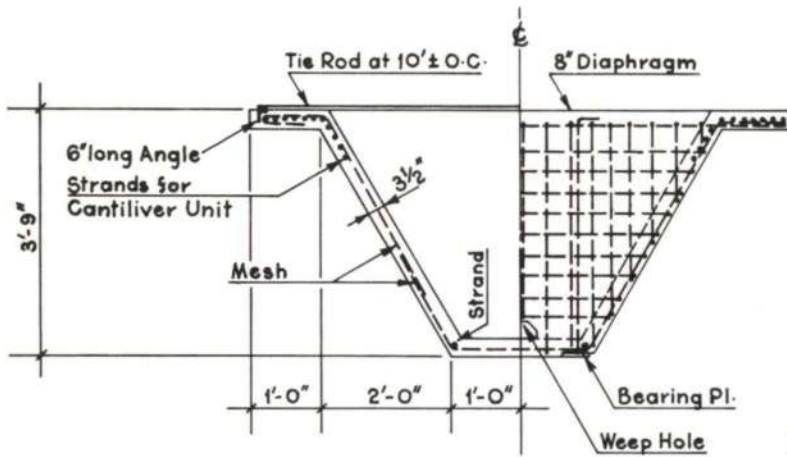
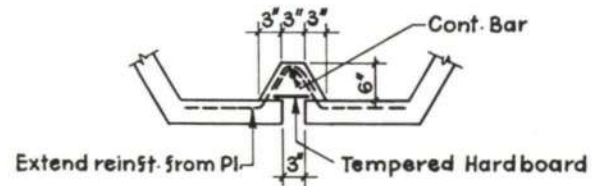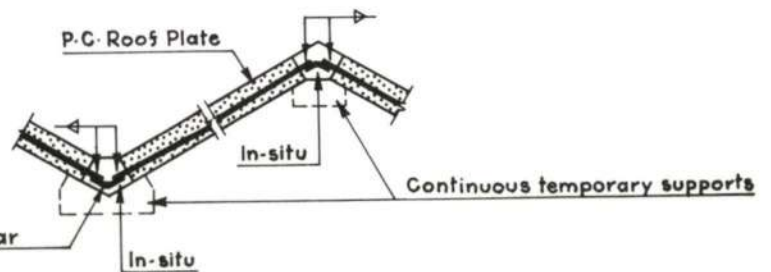**Fig.13a** Typical section and section at support

Fig.13b

Figs.13 a-c
Precast details

Fig.13c

SPAN/DEPTH RATIOS
Span to depth ratios vary considerably depending on the roof cross-section and whether it is reinforced concrete or prestressed concrete. Reinforced concrete roofs generally have span to depth ratios in the range 10 to 20, while prestressed concrete ones go up to about 30, though to achieve the latter, fairly narrow bays seem necessary. Some interesting facts and figures on different folded plate roofs as built are shown in tabular form.

DEPROPPING
Careful consideration should always be given to depropping and instructions given to the contractor. The problem is to ensure that full interaction between plates occurs as designed and that the proper deflections occur. Much will depend on the nature of the structure and each case should be considered on its merits.

CONSTRUCTION IN STEEL
In some buildings steel and concrete are valid competitors for constructing folded plates. Steel is generally cheaper where no special ceiling is required and on smaller jobs where little repetition of formwork can be achieved. It also depends on the shape of the roof, complicated cross-sections often being more suited to concrete. We must also remember that steel corrodes and requires maintaining. To my mind, cheap trussed steelwork in light rolled sections is seldom other than mundane in appearance. If the latter is important, such steel may well be better hidden, e.g. Modern Motors, Bulawayo. On the other hand, careful detailing and bold simple sections, e.g. welded tubes, can provide very elegant steel structures. Cost will of course need scrutiny in all cases.

ERECTION OF STEELWORK
The plates are generally of triangulated trusswork, multibay Vee-section roofs being typically made as interacting Warren girders (fig. 14a) though there are many ways of forming each plate. The interested reader may well refer to Architectural Design [6], which includes a resumé of some lattice shells designed by us. Depending on the size of each plate and available cranage, individual plates may be assembled on the ground and lifted up in one piece. Where this is not feasible the truss members are erected piece by piece, the procedure being as follows:

1. The top and bottom chords are positioned and propped.
2. The laterals and diagonals are lifted and bolted to them.
3. Ties are fixed in position.
4. Valley props should not be removed until at least one bay ahead has been completed and connected to it. Otherwise the truss self-loads will not be supported by full folded plate action and improper deflections will result. The depropping procedure should always be carefully considered.
5. Purlins, cladding and other trades follow behind the trusswork.

It is important to specify sufficient camber in this type of construction, both for visual and drainage reasons.

WATERPROOFING, DRAINAGE & INSULATION OF STEELWORK
Some form of corrugated sheeting, asbestos, galvanized iron, or aluminium is commonly used on folded plate steelwork. Where a light lattice steelwork has been used, stiffened by a timber membrane, flat sheet metal is generally used for weathering [6].

Rainwater is collected in the valleys in gutters often specially made in mild steel. Vertical adjustment should be provided in the gutter fixings to ensure adequate fall, over and above that provided by the structural camber where necessary (fig. 14b).

The thin weatherproofing membrane will often demand some form of backing as insulation and this may sometimes be applied as a layer immediately underneath the weathering, e.g. timber sarking, or as a suspended ceiling below the steelwork. The solution will depend largely on architectural, visual and cost requirements. A suspended ceiling may disguise the structural roof shape or hide it altogether from inside and good reasons would be necessary to justify this, to face possible charges of gimmickery! Voids left between the ridges and a suspended ceiling could, of course, provide good space for trunking etc.
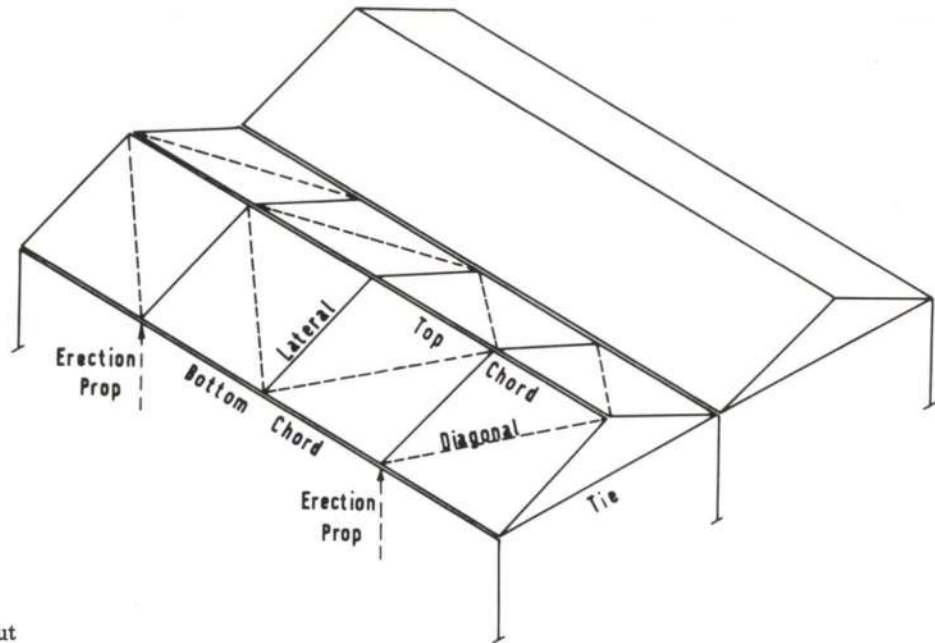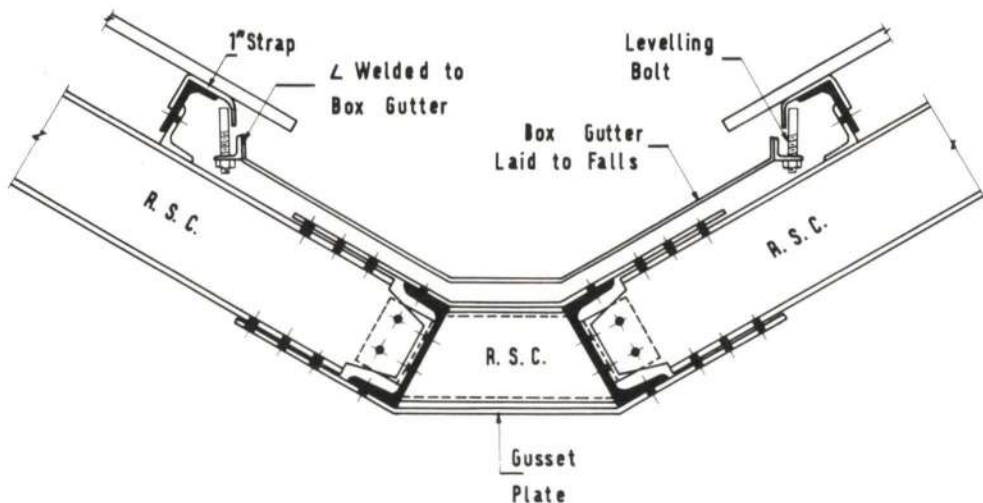
Fig.14a
Typical steelwork layout

Fig.14b
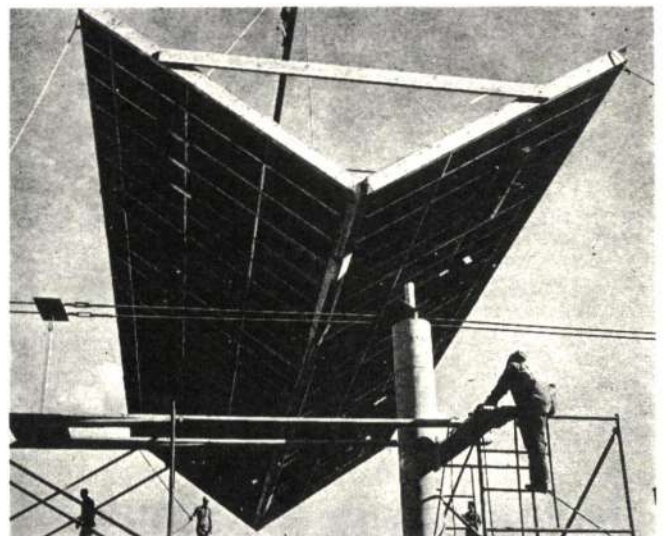Valley details

Figs. 14 a-b
Steelwork

MISCELLANEOUS

A common problem is that raised by the architectural
requirement of roof lighting. An obvious solution is a
north-light roof. In other roofs, if the architect dislikes
diagonals across the glazing, a vierendeel girder may be
feasible. The tie force is generally provided by a simple
steel angle at valley level along the column line.

Columns in steel or reinforced concrete at the ends of each
valley provide holding-down bolts for the steelwork with
normal provision for tolerance.

As usual in steel construction, thought should be given to
maintenance when detailing the structure. Accessibility
for repainting should be considered in relation to cladding
and other fixing.

CONSTRUCTION IN TIMBER

Timber, with its high strength to weight ratio, is ideally
suited to prefabrication. Light, readily-handled panels
can be made on the ground or in a factory and easily bolted,
nailed or glued together in place.

Despite its favourable strength to weight ratio, which makes
it so good for long span beam and trusswork, timber does
not yet readily make long span folded plates, probably
because of difficulties with tensile connections. Medium
spans, however, are quite feasible. Each timber plate

Fig.15
Prefabricated roof unit in timber, 67 ft. span
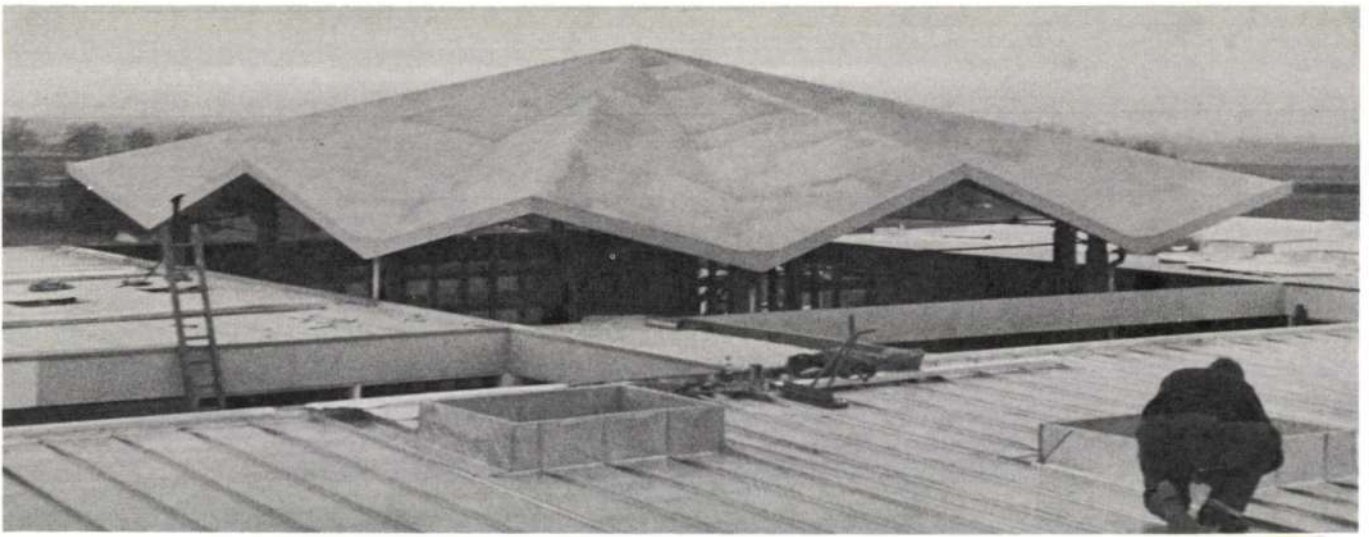(produced with permission from
Engineering News Record)

may be made as a truss or a so-called stressed-skin.
Figure 15 shows a prefabricated unit being lifted into
position; it is 50 ft. wide and spans 67 ft. and consists of
$\frac{3}{4}$ in. plywood panels on 8 in. x 2 in. framing at 2 ft.
centres. $\frac{5}{8}$ in. gypsum plasterboard was nailed to the
underside after erection. This was for a warehouse in
Denver, Colorado, about 50 such units being required in all.
Very elegant folded plate roofs can be made in timber.
A non-prismatic one of which we should be very proud is
Dothill Girls' Infant School (figs. 16, 17). With this sort
of shape the architect has all sorts of possibilities in
spatial treatment and exposed natural timber.

Dothill School roof is 47 ft. diameter in plan, made up of
16 triangular plates. Each plate was prefabricated from
two skins of $\frac{3}{8}$ in. plywood, attached to 4in. x 1$\frac{1}{2}$ in. framing
at 16 in. centres. The panels were bolted together in
position through the access slots visible in the photograph.
Waterproofing was effected with 22 gauge aluminium flat
sheeting (ALLSTRIP).
The fabrication and erection proved very straightforward
for the contractor.

Fig. 16 above
Dothill School, Shropshire
Assembly hall roof
(Photo: S.S. Heighway)

Fig. 17 below
Dothill School, Shropshire,
view showing bolt pockets
(Photo: S.S. Heighway)
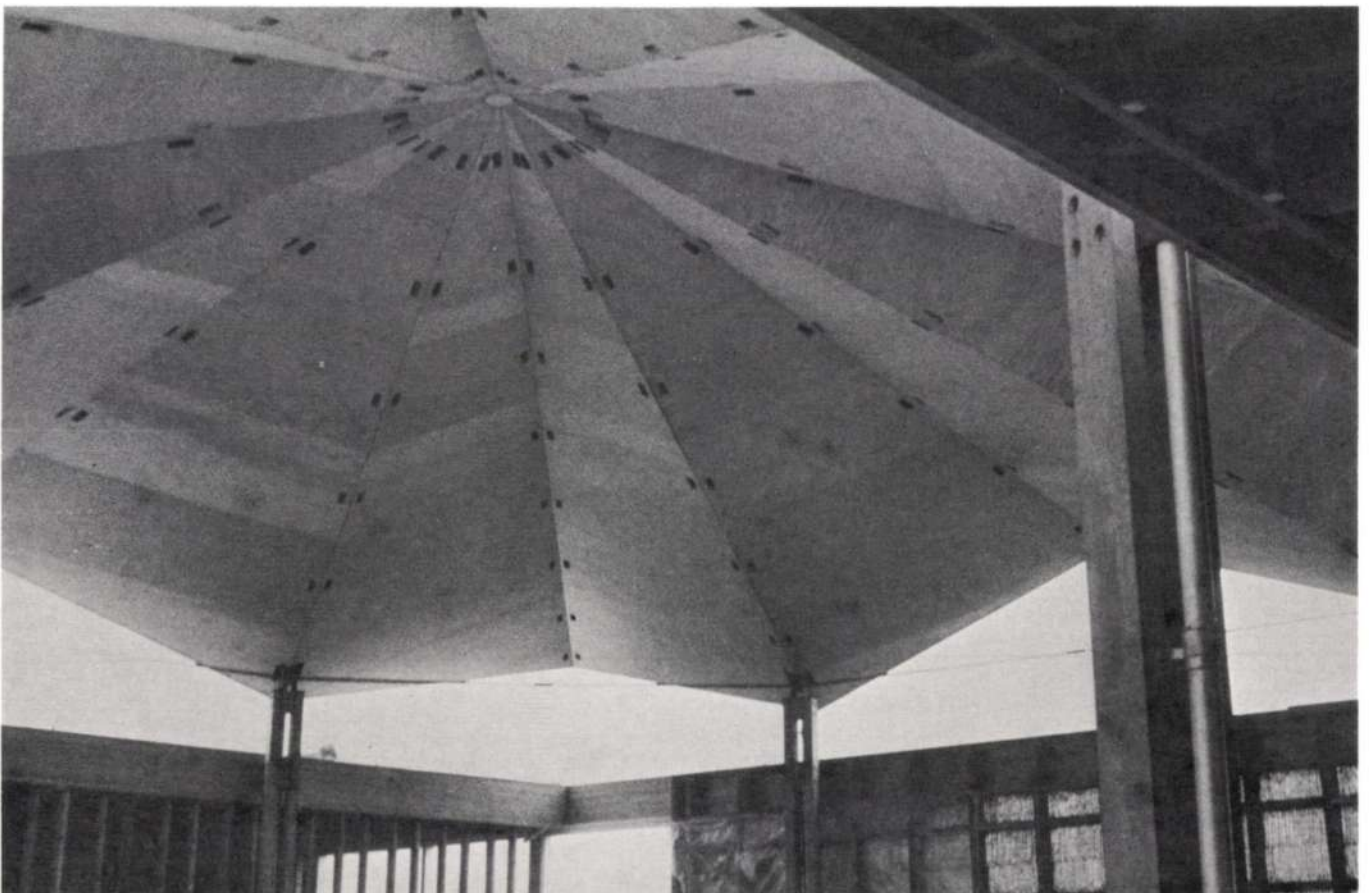
CONCLUSION

Folded plate construction gives us a very wide range of roof types, from which we can produce economical, functional and aesthetically pleasing roofs for a variety of types of everyday buildings. Perhaps we in Britain could profitably follow the trend set by some of our overseas friends, and use this medium more often?

ACKNOWLEDGEMENTS

I am very grateful to Ove Arup & Partners for the opportunity to write up this interesting subject, and also to various helpful Arupians from different offices who have so readily given advice and information for this article.

Credit is due to Lionel Francis and Godwin Gonzales for drawing some of the diagrams.

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Fenchurch Street

John Anderson

INTRODUCTION

In June 1965 we were commissioned by Messrs. Trollope and Colls who are the contractors of the Kleinwort-Benson Building at 13-23 Fenchurch Street, London, to advise them on
(i) the feasibility
and (ii) the method of construction
of the post-stressed concrete umbrella substituting the original high yield steel umbrella at the top of the building.

STRUCTURE

The overall structure consists of a central core, 45 ft. square, extending 290 ft. above ground level, supporting an umbrella system (fig.1a) from which the external edges of 22 light steel construction floors are suspended by means of structural steel hangers (fig. 1b). The core was constructed using a sliding shutter.

The main supporting floor, the 23rd, was originally designed in BS 968 steel. Mild steel had been considered but rejected on the grounds of delay in supply. During fabrication serious web defects were found after welding the larger universal column sections. Attempts to eradicate these defects proved to be unsuccessful and since the anticipated time to fabricate the various members from BS 968 plate was considered excessive it was decided to redesign the 23rd floor in reinforced concrete. This redesign necessitated a completely different approach to the method of construction and this report only deals with that aspect of our commission.

INVESTIGATION

Four possible methods were considered:

1. Erect the original BS 968 frame and strengthen it as necessary by prestress or casing.
2. Erect the floors upwards from ground level and then build the umbrella structure.
3. Erect a tubular scaffold and cast the umbrella structure on top of it.
4. Erect a structural steel platform at the 23rd floor level, suspended from the core, and construct the floor on it.

Method 1. This solution was rejected as too involved on a number of counts, particularly the redesign necessary.

Method 2. This solution was rejected mainly on the grounds that the banking hall completion would be considerably delayed.

Method 3. Calculations showed that movements of up to 7/8 in. could be expected due to temperature changes. In addition elastic shortening under the concrete load could amount to about 3/8 in. whilst further shortening due to give in the connections might occur. Considerable shoring of the ground floor would be needed to transfer the loads to the basement slab. For the above reasons it would be necessary to introduce a high degree of flexibility into the 23rd floor at the core until the concrete could be lifted off the scaffold. Within these limits a feasible scheme could be developed.

Method 4. A platform based on the Bailey Bridge principle was designed and methods of suspension investigated. This appeared to be workable and the movements due to loadings appeared to be controllable to a higher degree than the scaffold. In addition, a rough cost estimate indicated that the platform would be considerably cheaper than scaffolding.

Method 4 was therefore adopted.

DESIGN

The layout of the Bailey components is shown in figures 2 and 3. This layout was determined more by the shape of the structure and sizes of the standard Bailey components than by the shear and moment resistances required. In most cases single panels would have been sufficient but since the four-way connections, pile yokes, were designed for double panels it was decided that the incorporation of a second panel would provide more stability.

The design of the supporting fixtures was considerably more difficult. Since the core had been completed before the redesign had been decided upon there were no built-in fixtures on which to attach the platform. In addition, the
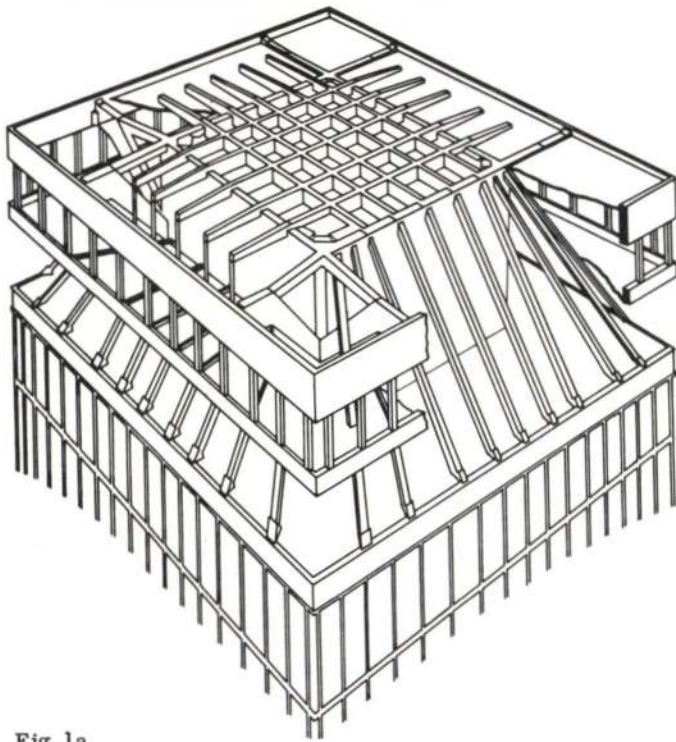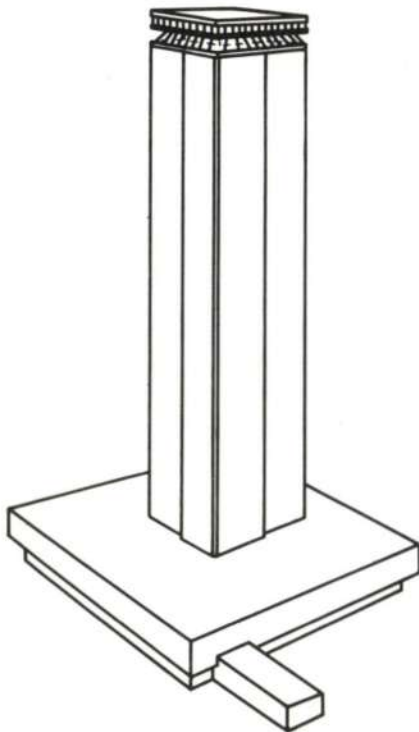
Fig.1a

Fig.1b

Figs.1a-b
Completed structure
Drawings by Godwin Gonzales

Fig.2
View of completed platform
looking west along Fenchurch Street
(photo: P.I.C. Photos Ltd.)

amount of reinforcement in the core walls was such that
a minimum of cutting and drilling was desirable.
It had been originally hoped to take the inner reaction of the
platform and 23rd floor on the core corbel but this proved
to be insufficiently strong and so suspension from the top of
the core was necessary for the entire loading.
The suspension of the platform from the top of the core
presented three separate problems. Firstly, the loads
from the vertical suspension rods $1\frac{1}{2}$ in. diameter mild
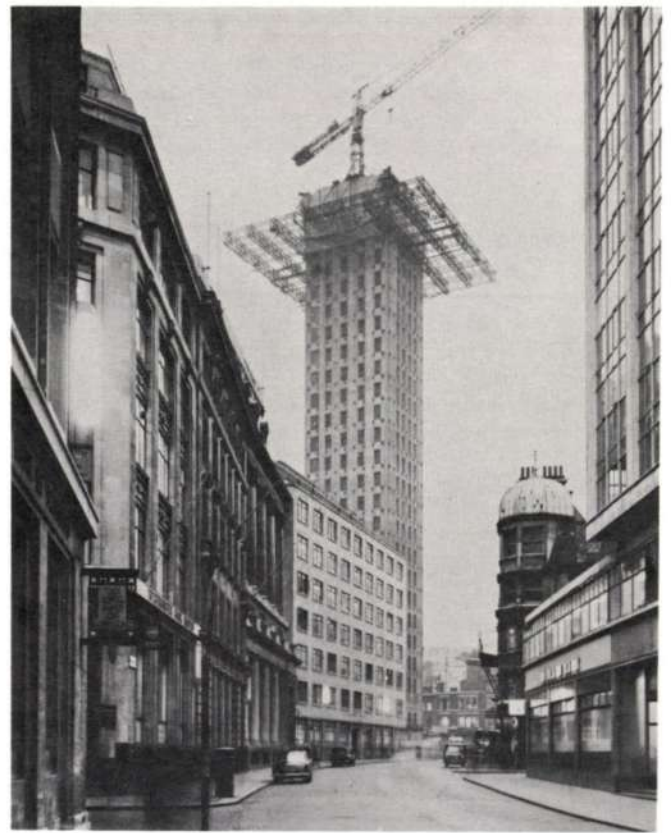steel from the inner edge of the platform along the side of

the core had to be taken over the top of the core wall on a
rocking beam, the inner end of which had to be tied down
inside the core. It so happened that there were a number of
rawlbolts on the inner face of the core that had been placed
to secure a crane for erecting the original steel frame.
These bolts however did not supply sufficient reaction in
themselves and had to be supplemented. A further
complication was presented by the fact that all four core
walls were of different thickness and had varying internal
wall layouts and therefore had to be considered separately.
Secondly the loads from the raking suspension rods, also
$1\frac{1}{2}$ in. diameter mild steel, from the outer edge of the
platform along the side of the core had to have fixings
designed to give a minimum outward reaction on the core
walls. This problem was solved by connecting opposite
fixings with $1\frac{1}{2}$ in. mild steel rods across the core. The
elastic strain of these rods under full load was estimated
to be of the order of 1/8 in. and it was decided to take up
this extension prior to loading by pretightening the rods.
In this way the only outward reaction to the core would be
as a result of friction between the fixing on top of the core
and its base plate.
Thirdly the loads from the vertical suspension rods on the
inner corners of the frame had to have a composite fixing
which would both act as a rocker beam and diagonal tie
across the core. In this case it was not possible to tie
down the inside end of a rocker beam to the core since
there were no bolts nor internal walls. This problem was
solved by designing a composite connection around a 14 in.
x 16 in. universal column section which extended diagonally
across the core. Holes were to be cut in the various
internal walls and the beams wedged through them. Where
the two diagonal beams met a cruciform joint was designed.
The suspension points can be seen in figure 4.
The connection of the suspension rods to the Bailey frame
presented two problems. Firstly the vertical rods. The
solution of this connection was comparatively simple and

was basically a base plate, through which the rods passed with nuts on the underside, welded to two channels which spanned across the chords of the pile yokes.

Secondly the raking rods. The solution of this connection was essentially the same as for the vertical connection, except the base plate was inclined at right angles to the raking rods. In order to overcome the horizontal thrust the base plate was bolted to the bottom chords of the pile yokes with eight high strength friction grip bolts.

FABRICATION

The fabrication of the various fixing units was subcontracted by Trollope and Colls to Lawrence Colebrook & Co. Ltd. All the materials were delivered to site and the fabrication carried out there. No major snags were encountered during the fabrication.

In order to ascertain the degree of workmanship Messrs. Sandberg were retained to carry out proving tests. Three tests were carried out.

1. Two pieces of 1 in. plate were butt welded and submitted for radiographic examination. No defects were visible apart from minor stop/start porosity.
2. Ultrasonic tests were made on site on some 6 welds selected at random. No defects were detected in any of these welds.
3. A 3 ft. length of suspension rod was threaded at each end and submitted for a tensile test. The rod was cut in two and each section tested to failure which in both pieces occurred on the threaded length. The failure loads were 44.68T and 44.13T giving an average of 35T/sq.in. The design stress is 8T/sq.in.

ERECTION

The erection of the Bailey frame and all fixings and suspension rods was carried out by Lawrence Colebrook using Trollope and Colls' tower crane. This was done in the following sequence:

1. 14 x 16 universal column sections diagonally across the top of the core. This operation was hampered by the necessity of threading the members through various holes in the internal core walls and through the crane gabbart. The cruciform joint was welded in situ.

Fig. 3
View of completed platform from underneath
(Photo: P.I.C. Photos Ltd.)

2. All other fixings on the core walls including the horizontal ties across the top of the core.
3. The collar section of the Bailey frame with suspension rods. This was raised in four pieces which were made up on the ground before raising.
4. The outer sections of the Bailey frame alongside the core. These also were made up on the ground into large units before raising.
5. The corner sections of the Bailey frame. As before these were raised in large units.

The erection took place during November 1965 and took some five weeks. During this time however a considerable amount of time was lost due to inclement weather in the form of snow and ice on the frame sections, making access both difficult and dangerous. The sequence of construction can be seen in figure 5.

When the erection was complete the entire platform was levelled. It was found impossible to do this by tightening the nuts on the suspension rods and was eventually effected by raising each suspension point to its level by the crane and tightening up the suspension rod nuts to hand tightness plus about $1/4$ turn. In this way the frame was levelled to $\pm 1/8$ in. at the suspension points with varying sags between due to the inherent looseness of the pin joints between panels. The outside edge of the frame was set $1\frac{1}{2}$ in. high to counteract the elastic extension of the suspension rods.

TIMBERWORK AND EDGE BEAM

In order to obtain a fairly uniform spread of load to the frame, a deck of 12 in. x 6 in. timbers was placed over the entire frame area and strapped to the Bailey panels. Around the outside of this was placed a walkway and guard rail some 7 ft. high covered with chicken wire and hessian. In addition, safety nets were suspended underneath the Bailey frame. The underside of some of the 12 in. x 6 in. timbers were packed with wedging to counteract the frame sag and bring the top surface level (fig. 6).

The edge beam was then brought up in sections and placed approximately to line and level on pre-levelled packing pieces. When placing was complete it was found that the $1/4$ in. sag allowance had been taken up whereas at that stage only $1/8$ in. should have been used. This was due to 'take up' and irregularities in the 12 in. x 6 in. timbers. The edge beam was then set to required line and level, a further $1/8$ in. packing being provided and after checking, the sections bolted together (fig. 7).

SHUTTERING TO FLOOR

During the placing of the edge beam the fixing of the soffit shuttering was begun. A grid of timber was placed on the 12 in. x 6 in. base timbers to cater for varying soffit levels and the soffit shape formed in $3/4$ in. plywood (fig. 7).

CONCRETE TO FLOOR

During the design it had been decided to omit as much concrete as possible until the permanent ties had the weight of the floor in order to lighten the platform and suspension fixings. In fact only two restrictions were found possible.

1. The outer half of the floor slab was not to be cast until all the necessary stressing was complete and the frame unloaded.
2. The rest of the floor slab and thrust beams were to be cast and suspended from the ties in two stages, the frame being unloaded between the stages. The first stage was to include only the floor alongside the core and the second stage the remaining four corner sections.

Floor casting therefore was carried out along these lines by normal methods. One further precaution was considered necessary. Owing to the fact that small extensions in the suspension rods would undoubtedly occur during the concreting operation it was felt if concrete was placed outwards

from the core there would be a tendency for the green concrete to shear along the line of the corbel. If however concrete was placed towards the core all extensions would have taken place when the corbel was reached and no shear would be imposed. (fig. 8).

TIES

As each area of the 23rd floor was concreted, shutters for the ties were erected and the concrete placed. When the desired strength was reached these were stressed in the sequence dictated by the restrictions mentioned above (figs. 8 and 9).

DISMANTLING

At the time of writing* dismantling has not begun. The following is an envisaged method:
1. When the floor has been taken on the hangers, loosen all the raking bolts by taking the frame weight with the derrick and undoing the lower nuts.
2. Working inside the Bailey frame, loosen all the soffit timber and remove.
3. Slide the 12 in. x 6 in. timber outwards, sling from the derrick and lower to the ground.
4. Dismantle the outer rim of Bailey panels and pile yokes in as large sections as possible and undo nuts on suspension rods. Lower to ground.
5. Similarly dismantle and remove panels between outer and centre runs of panels.
6. Using winches set up on 23rd floor and holes in the slab, dismantle and lower remainder of frame in as large sections as possible.

Architects: City of London Real Property Co. Ltd.
Consulting Engineers: John Mason and Partners
Contractor: Trollope and Colls Ltd.

* Editorial note: September 1966

Fig. 4
Top of core showing suspension details
(Photo: Fox Photos Ltd)

Fig. 5
Underside view of platform during erection
(Photo: Fox Photos Ltd.)
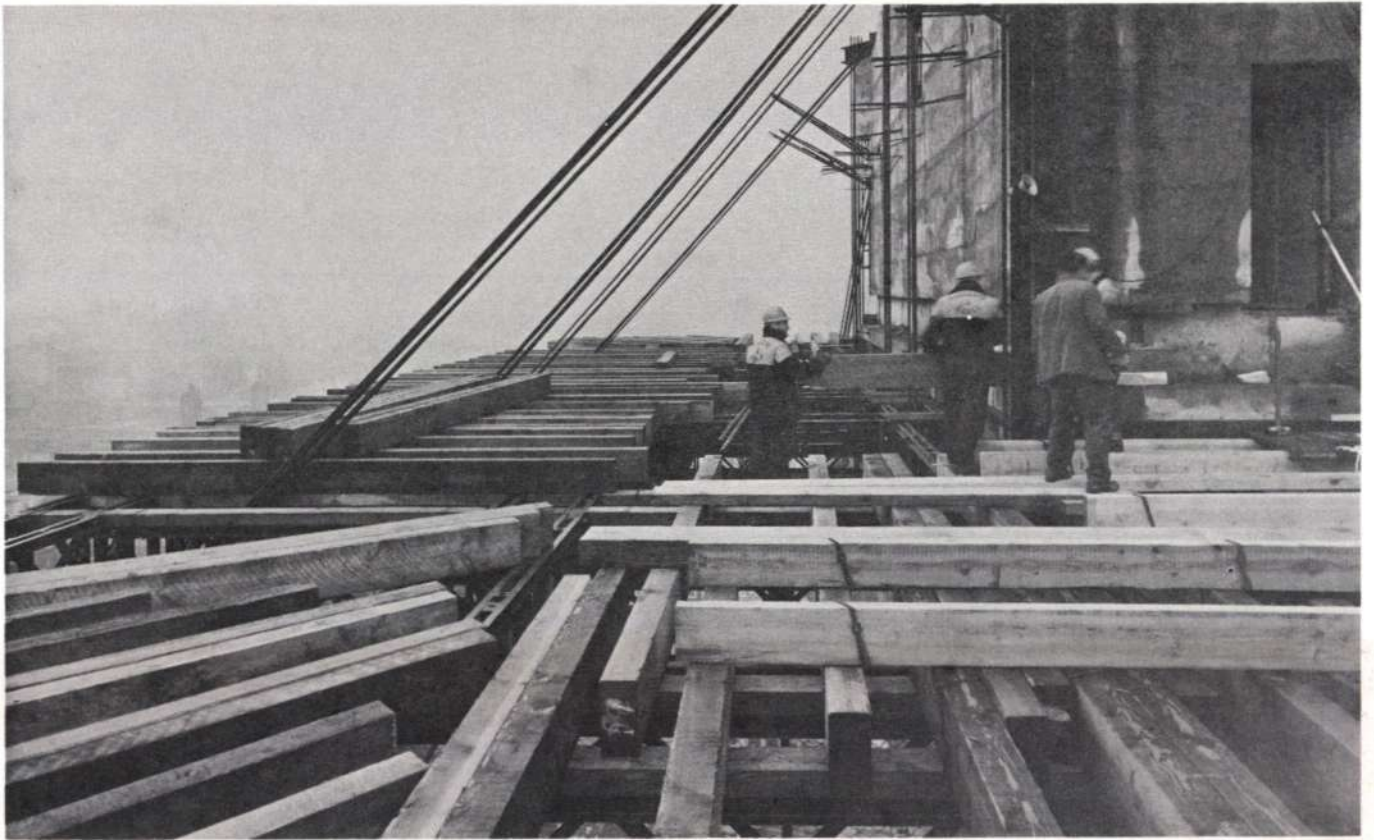
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Fig. 6
Timber layout on platform
(Photo: Fox Photos Ltd.)
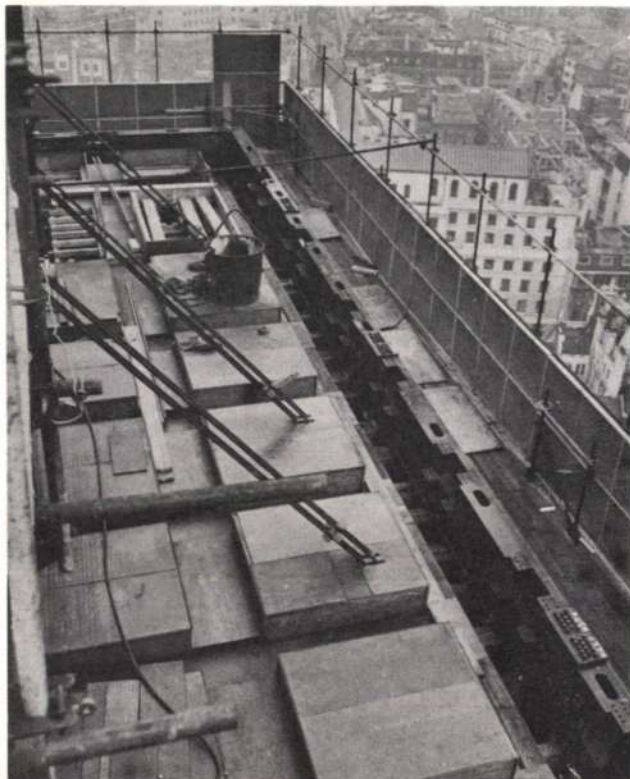
Fig. 7
Edge beam and floor formwork in place
(Photo: Fox Photos Ltd.)

Fig. 8
Shutters for ties
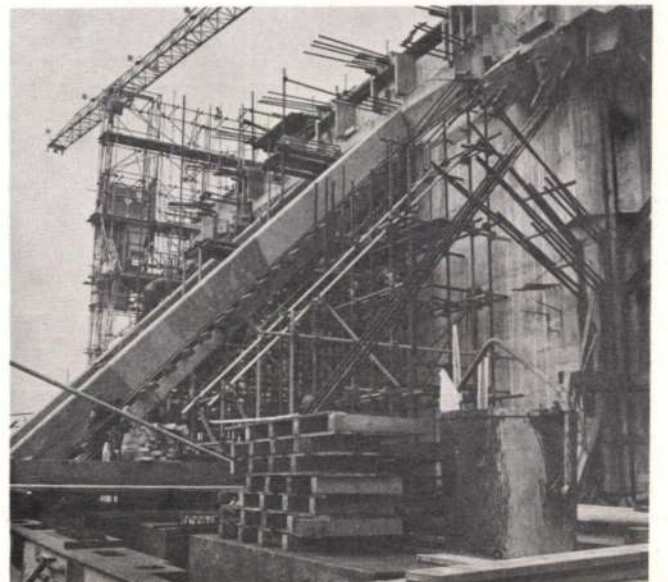(Photo: Fox Photos Ltd.)

Fig. 9
Concreted ties

8

7

9