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Contents

The photograph on the front cover was taken by Max Dupain and that on the back cover by G. Wood.

*This paper, which opens this issue of The Arup Journal, will be discussed at the October Technical Staff Meeting. Everyone is welcome

Shear resistance of flat slabs around column heads*

C. M. McMillan

INTRODUCTION

Existing methods of estimating the shear capacity of flat slabs are generally based on comparing a shear stress calculated using an estimated shear perimeter, with an allowable shear stress, related to the strength of the concrete. Codes of practice differ widely in their definition of the shear perimeter and the allowable shear stress. The designer working to C.P.114:1957 is faced with a dilemma if the calculated shear stress is near the permissible value: while no shear reinforcement is required below it, it would appear that the entire shear force must be carried by reinforcement if the limit is exceeded. No account is taken of the effect of the top bending reinforcment over the column head in resisting shear forces, although numerous tests have revealed its significance.

Many test results have been reported in recent years, (2), (3), (4) and empirical equations have been proposed relating the principal variables, namely the slab depth, column size, quantity of reinforcement, concrete strength and steel strength, to the punching strength of the slab.

This note is based on a paper by Yitzhaki in the *American Concrete Institute Journal* of May 1966 (1) in which a new relationship is proposed and checked against the experimental work of its author and many others. Its appeal lies in its simplicity and its apparent agreement with observations.

THEORY

Two types of failure are possible in a flat slab in the vicinity of a column head: a flexural or a punching failure. The problem is to estimate the ultimate strengths of the slab, P_{flex} and P_{pun} respectively, for the two types of failure. A design for which the two are equal will be termed balanced.

Flexural strength

The flexural strength can be estimated without difficulty by yield line methods, the yield pattern comprising a circumferential or polygonal crack associated with a large number of radial cracks. The flexural strength P_{flex} is then dependent on the column size, but for most practical cases a good approximation, allowing for some strain hardening in the reinforcement, is

$$P_{flex} = 8\ m_p = 8\ pf_y jd_1^2 \qquad (1)$$

where P_{flex} is in lbs,
m_p = the yield moment per unit length of the slab for hogging moments over the column,
p = the reinforcement ratio, i.e. the ratio of area of reinforcement to concrete cross-sectional area,
f_y = the yield stress of the tensile reinforcement in lb. per sq. in.,
d_1 = the effective depth in inches
and j = the lever arm ratio.

In accordance with American custom, the lever arm ratio is given as $j = 1 - \frac{pf_y}{2f_c}$
where f_c = the concrete cylinder strength.

For the purposes of this paper it will be assumed that the cube strength $u = 1.25\ f_c$

$$\text{so that} \quad j = 1 - 0.625\ \frac{pf_y}{u} \qquad (2)$$

Punching strength

Many empirical equations have been proposed to define the punching strength of flat slabs. A feature of the relationship proposed by Yitzhaki is the significance it places on the quantity of bending reinforcement over the column. The nature of the effect is illustrated in Fig. 1. The load is resisted by friction between the slab wedges and the column faces. Punching failure could result from either

1. compressive failure of the concrete in the wedge, or
2. the vertical frictional force at the column face being inadequate.

The latter is raised by an increase in the compressive force there, which in turn makes an increased amount of tensile reinforcement necessary. Tests have revealed that the compressive strength of the concrete itself is not exhausted at punching failure, because similar slabs with more reinforcement show higher punching strengths.

The equation proposed for calculating the punching strength of a slab is

$$P_{pun} = \frac{8jd_1^2(149.3+0.164pf_y)}{(1+0.5^r/d_1)} \qquad (3)$$

where P_{pun} = the punching strength in lbs,

and r = the column size in inches.

For square columns r is the side length, while for other shapes Yitzhaki suggests using the side length of a square of equal area. It would, however, seem that to take r as the side of a square of equal perimeter might be more logical.

Equation (3) is based on all the available test data, and the statistical evidence suggests that it is at least as satisfactory as the more complex expression proposed earlier by Moe (3) and later accepted by the ACI-ASCE Committee 426 (2).

ASSUMPTIONS AND LIMITATIONS

Because of the empirical nature of the above relationships, care must be taken not to extrapolate them to applications outside the range of the test results from which they were derived. Only results for slabs symmetrically loaded, with circular, square or near-square columns and isotropic reinforcement, are reported. Caution and judgement are necessary where the reinforcement is anisotropic or a rectangular column with widely different side lengths is encountered. Where moments have to be transferred through the slab-to-column connection, reference should be made elsewhere (2).

On the other hand a wide range of concrete cylinder strengths between 1780 and 7330 lb. per sq. in. and reinforcement strengths (pf_y) between 483 and 1724 lb. per sq. in. are covered.

COMPARISONS WITH CURRENT PRACTICE

On the basis of equation (3) the nominal shear stress at failure on a shear perimeter a distance d_1 from the face of a square column would be

$$f_s = 149.3 + 0.164\ pf_y.$$

This implies a limiting shear stress independent of concrete strength, and related instead to the quantity of reinforcement over the column.

If, on the other hand, the nominal shear stress is calculated at a distance of $\frac{1}{2}d_1$ from the column face, then at failure

$$f_s = (149.3 + 0.164\ pf_y)\ \frac{2+{}^r/d_1}{1+{}^r/d_1}$$

For ratios ${}^r/d_1$ between 0.5 and 2.7, this gives the ultimate shear stress as between 190 and 250 p.s.i. for an unreinforced section. A practical quantity of 0.8% of high tensile bending reinforcement would increase this by 50%.

It can be seen from equation (3) that the punching strength per unit length of column perimeter increases with a reduction of column size. This is consistent with the theory, because the compressive stresses producing wedging action near the bottom of the slab section are greater the smaller the column.

GRAPHICAL REPRESENTATION

In Fig. 2 the parameter $P/(8jd_1^2)$ is plotted against the reinforcement strength pf_y. For flexural failure, according to equation (1)

$$\frac{P}{8jd_1^2} = pf_y \qquad (4)$$

giving the line a-b. For punching failure, from equation (3)

$$\frac{P}{8jd_1^2} = (149.3+0.164\ pf_y)\ (1+0.5^r/d_1) \qquad (5)$$

The latter leads to the set of straight lines for different ${}^r/d_1$ ratios.

A balanced design occurs when the reinforcement strength corresponding to a given load P is defined by the line a-b. The ${}^r/d_1$ ratio appropriate to the balanced condition is then fixed. It is apparent from Fig. 2 that for a chosen ${}^r/d_1$ ratio, increasing the reinforcement strength would raise P_{flex} considerably more than P_{pun}.

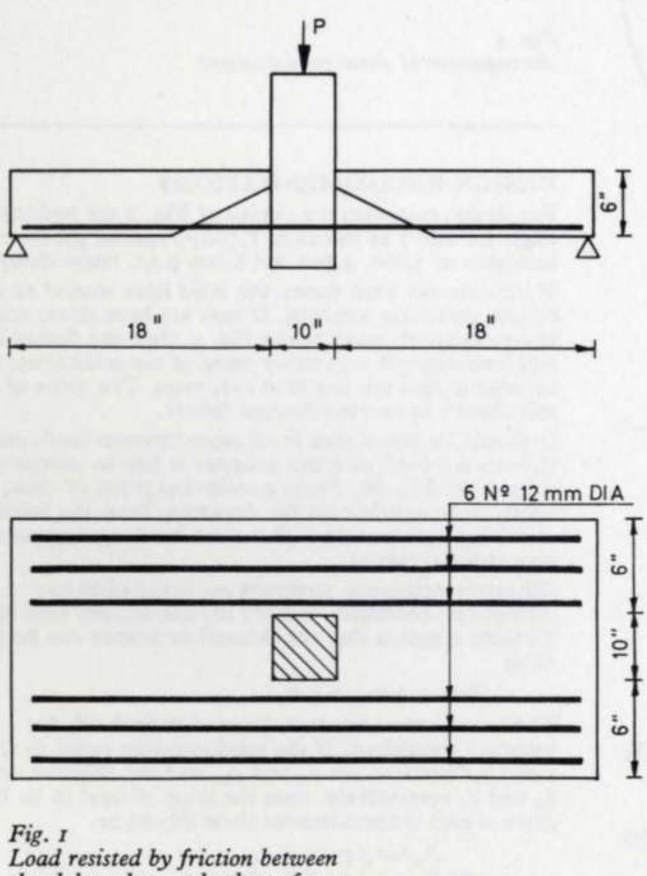


Fig. 1
Load resisted by friction between the slab wedges and column faces

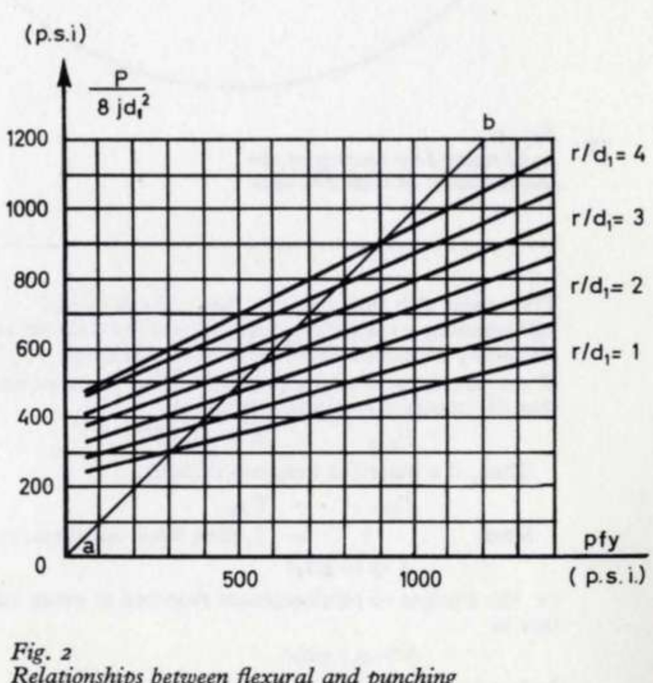


Fig. 2
Relationships between flexural and punching strengths and reinforcement strength

This demonstrates that the economical way to increase P_{pun} is to increase r/d_1 and not pf_y. Another way of increasing P_{pun} relative to P_{flex}, namely by inclined reinforcement, will now be discussed.

EFFECT OF DIAGONAL REINFORCEMENT

Further tests revealed how the punching strength of a slab with a given quantity of reinforcement may be increased by bending down a portion of that reinforcement in the form of diagonal shear bars.

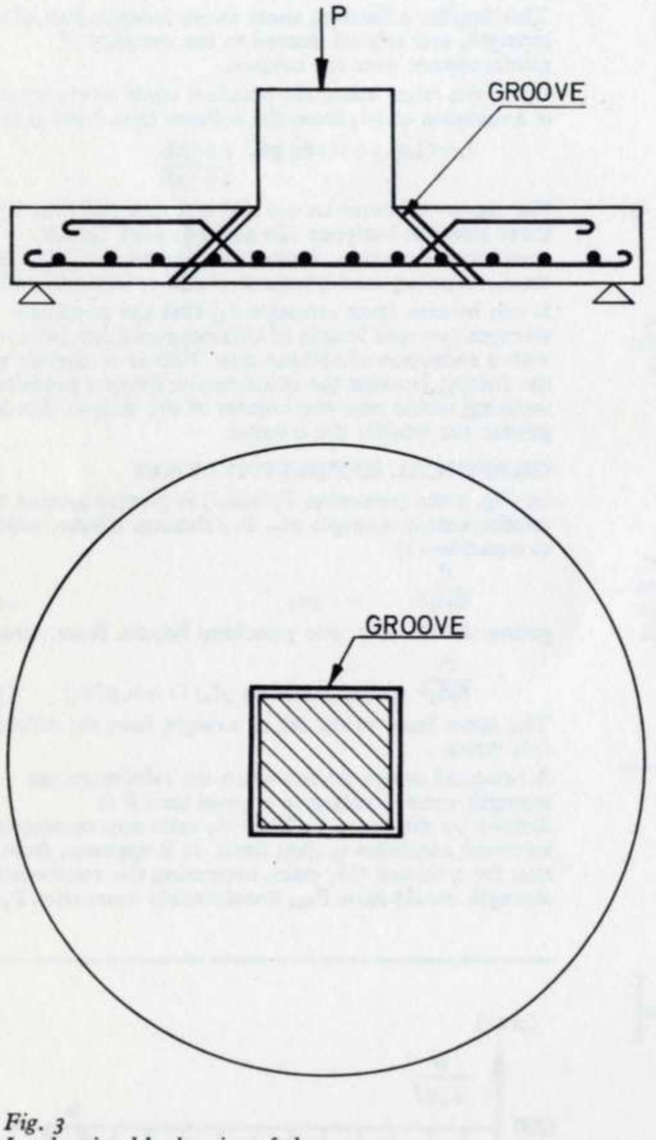


Fig. 3
Load resisted by bearing of the reinforcement in both directions

The model slab illustrated in Fig. 3 is capable of withstanding a substantial load despite the column being physically separated from the slab by a groove.

If the punching strength is calculated on the assumption that the concrete does not contribute,

$$P_{pun} = A'f_y / \sqrt{2}$$

Then, if a balanced design is desired,

$$P_{flex} = P_{pun}$$

Since $j = \frac{7}{8}$, then from equation (1),

$$A' \doteqdot 10\ pd_1^2 \qquad (6)$$

i.e. the amount of reinforcement required at every column face is

$$A'' = 2.5\ pd_1^2 \qquad (7)$$

In fact it is claimed on the basis of tests that if this quantity of reinforcement is bent down, the punching strength of the slab is increased by 50%. The more conservative figure of 33% is, however, recommended by Yitzhaki for design, and is included in the design charts later.

Because the intensity of hogging moment falls off very rapidly away from the column, it can be shown that provided the reinforcement is bent down according to Fig. 4, no additional bars are required. The portion A′ of the main reinforcement can be bent down without reducing P_{flex}.

It should be noted that the form of the diagonal reinforcement produces concentration of bearing stresses in the concrete at the bends. The recommendation of a limiting radius of bend of (10 x bar diameter) is therefore made. A further practical recommendation of the ACI Committee on Shear (2) is that inclined reinforcement is not desirable for slabs less than 10 in. thick.

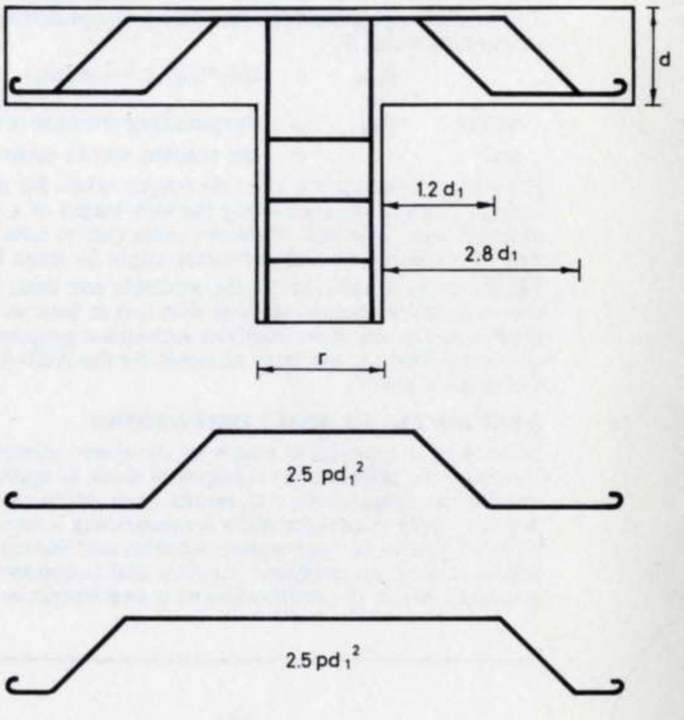


Fig. 4
Arrangement of shear reinforcement

DESIGN RECOMMENDATIONS

For design purposes the curves of Fig. 2 are replotted in Figs. 5, 6 and 7 as values of $P/(8d_1^2)$ against pf_y for cube strengths of 3,000, 4,500 and 6,000 p.s.i. respectively.

If bars are not bent down, the solid lines should be used to give punching strength. If bars are bent down according to equations (6) and (7) and Fig. 4, then the dotted lines with ordinates of 1.33 times those of the solid lines, should be used to find the required r/d_1 ratio. The value of pf_y is still chosen to prevent flexural failure.

It should be noted that in all cases ultimate loads and stresses are used, and the designer is free to choose a suitable load factor. From a statistical point of view, there seems to be justification for departing from the balanced condition, and using a higher load factor against punching than flexural failure.

When the resistance moments are unequal in two orthogonal directions, and are m_x and m_y per unit length, Yitzhaki suggests that the flexural resistance can be taken to be

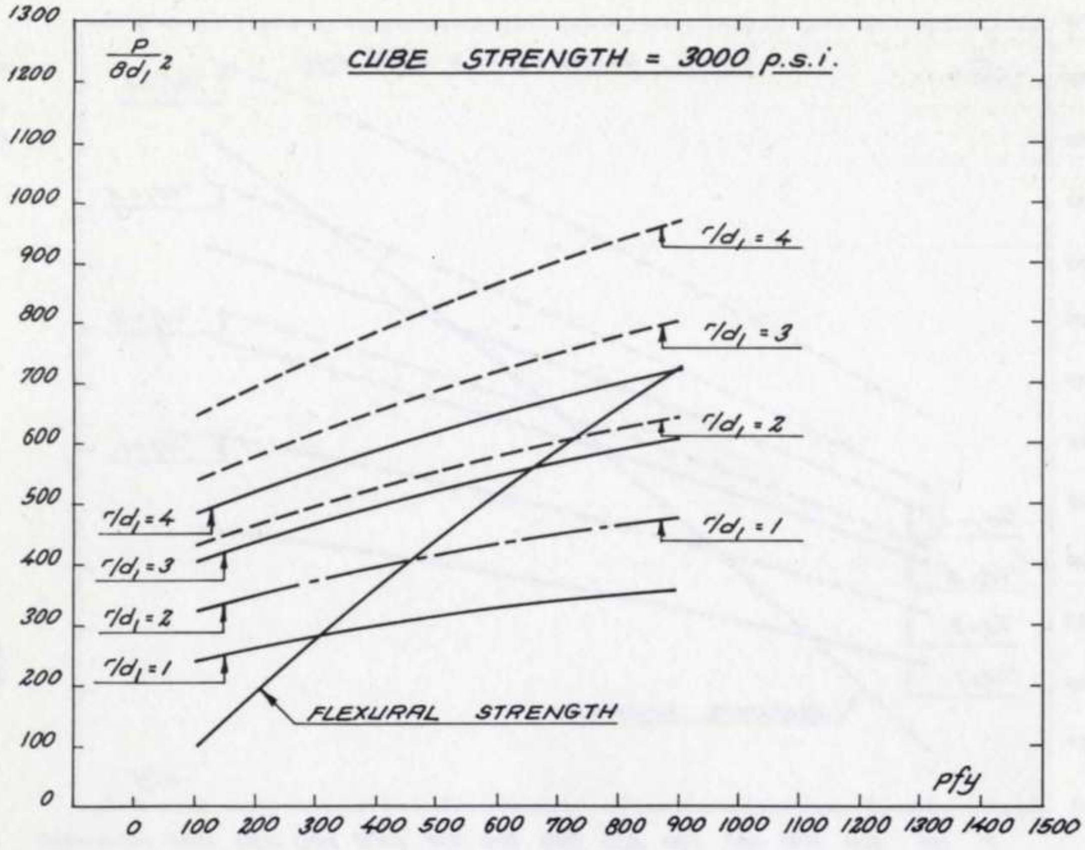
$$P_{flex} = 4\ m_x + 4\ m_y$$

The same curves can then be used to find r/d_1 for balanced conditions. If the reinforcement ratios in the x and y directions are p_x and p_y, and the effective depths d_x and d_y respectively, then the areas of steel to be bent down at each column face for shear should be

$$\left.\begin{array}{l} A_x = 2.5\ p_x\ d_x^2 \\ \text{and } A_y = 2.5\ p_y\ d_y^2 \end{array}\right\}$$

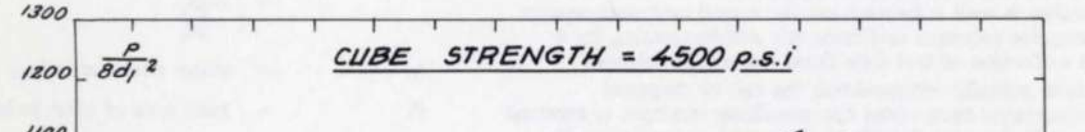
in the two directions respectively.

Caution should be exercised if m_x and m_y differ greatly.



——— Slabs *without* shear reinforcement ------ Slabs *with* shear reinforcement

Fig. 5
Design curves



——— Slabs *without* shear reinforcement ------ Slabs *with* shear reinforcement

Fig. 6
Design curves

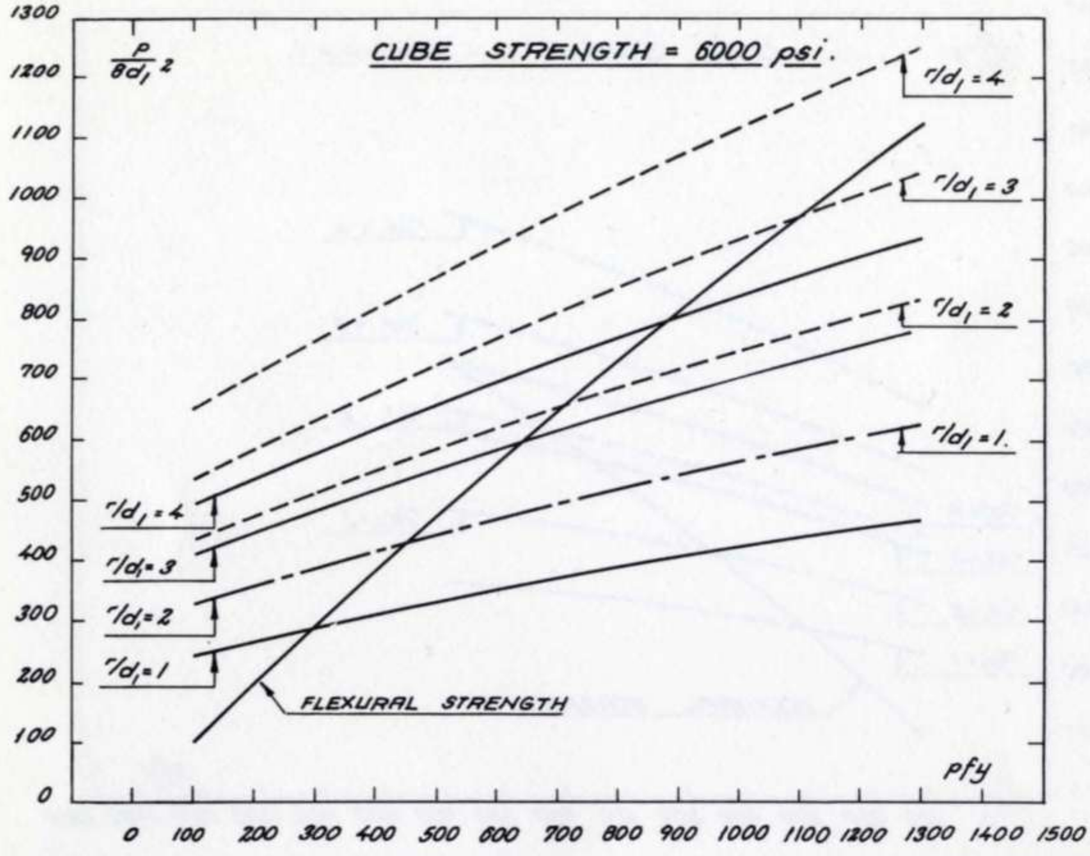


Fig. 7
Design curves

CONCLUSIONS

Yitzhaki's paper is seen as a useful development in the design of flat slabs for shear, since it defines the punching strength in terms of all the most important parameters instead of introducing meaningless shear stresses. Its reliability is well substantiated for round and near-square rectangular columns and isotropic reinforcement, by a large collection of test data from numerous sources.

Yitzhaki actually recommends the use of diagonal reinforcement even when the punching strength is ensured without it, because it adds to P_{pun} without reducing P_{flex}. This, however, would seem unnecessary in view of the amount of bar bending involved.

No suggestion is made by Yitzhaki as to the width of the reinforcement band of intensity p, but it is felt that it should extend at least the slab depth beyond the column face on each side.

The curtailment of reinforcement away from the column can be determined by conventional yield line methods.

A design carried out in this way has the advantage of concentrating reinforcement over the column head, as recommended by Brotchie (5), for the purpose of improving the elasto-plastic behaviour of flat slabs. In that reference are also given suggestions as to the overall distribution of top reinforcement along the column lines.

LIST OF SYMBOLS

d_1	=	effective slab depth
u	=	concrete cube strength
f_c	=	concrete cylinder strength
f_y	=	yield stress of reinforcement in tension
m_p	=	ultimate slab moment per unit length in lb. units
P	=	the column load at failure in lb.
P_{flex}	=	the ultimate load of the slab for a flexural failure around the column in lb.
P_{pun}	=	ultimate load of the slab for punching failure in lb.
r	=	column size in inches
	=	side length of a square column of equal area (or perimeter)
p	=	the reinforcement ratio, i.e. the ratio of area of reinforcement to concrete cross-sectional area
j	=	lever arm ratio
	=	$1-\frac{pf_y}{2f_c}$
f_s	=	shear stress at failure
A′	=	total area of steel to be bent down
A″	=	area of steel to be bent down on each column face
m_x, m_y	=	ultimate moments per unit length in two mutually perpendicular directions
p_x, p_y	=	reinforcement ratios associated with m_x and m_y respectively
d_x, d_y	=	effective depths associated with m_x and m_y respectively.

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Loughborough University of Technology; growth change and grid disciplines

David Thomas

INTRODUCTION

In an age of rapid scientific and technological progress obsolescence in building is an increasingly important and badly neglected problem. Functional and mechanical obsolescence are the most serious forms of obsolescence in building. Structural obsolescence is generally less of a problem and, given the right materials, buildings will remain structurally sound long after they are functionally and mechanically out of date.

There can be few buildings 50 or even 25 years old which are still used as they were originally intended to be used, but as people are fairly adaptable we get by—at a cost to our environment and efficiency.

The problem of functional obsolescence becomes most acute when buildings are designed to suit a particular use. The more closely a building is tailored to its function, whatever it is, the more quickly it will become out of date. Similarly with the mechanical services, although the increasing rate of technological development in the field of services may cause equipment and built-in services to become obsolete long before the plan form or fabric of the building.

In addition, mechanical equipment deteriorates much more rapidly than the fabric, and requires constant maintenance and eventual renewal.

The case, therefore, for designing buildings which will allow the subtraction, addition and renewal of mechanical services is overwhelming. Equally overwhelming is the case for designing buildings which will allow re-arrangement of the plan. The ability of a building to grow is as important as the ability to change in avoiding obsolescence. Growth pressures will vary according to the type of accommodation, but almost all building types would benefit from the ability to expand.

The first growth pressures usually result in a more intensive use of the existing accommodation, emphasizing the need for adaptability. Later pressures demand the addition of structural units of growth and finally, when sites are saturated, a complete new building.

In designing for growth and change it is, therefore, necessary to have a high degree of adaptability in the whole and in the parts.

Flexibility and adaptability are terms used in discussion about growth and change. Flexibility tends to be used where the building is designed so that a wide range of choices is instantly available to the user. To achieve this end the building is necessarily expensive since all predictable requirements are built-in. Adaptability tends to be used where a building is designed to be capable of being almost instantly adapted to new requirements. A condition of adaptability is that it must allow change with the minimum of inconvenience to the users.

Growth in large structural units can be allowed for in terms of adaptable buildings, but the enormous economic consequences of planning for growth in terms of flexibility are generally inhibiting.

LOUGHBOROUGH

In Loughborough we have tried to design buildings to meet the needs of both growth and change.

The lack of a definite brief at the outset—an intelligent refusal to hazard a guess at future developments in teaching disciplines by the client—made the development of such a building the only viable solution.

Our proposals are set out in the Master Plan Report, but briefly the solution was to propose a series of dimensional relationships realised as grid networks, giving a discipline within which the various parts can be related to each other and to the whole.

The use of grid networks in the design of buildings is not new, and some of the more obvious advantages are:

a.
The dimensions of a building can be more easily co-ordinated and rationalised, allowing factory or off-site produced components to be fitted together with the minimum of site labour, cutting and jointing.

b.
Sites can be planned to allow future growth within a pre-determined pattern based on a building unit of growth.

c.
Buildings can be planned using interchangeable parts, which will allow later re-planning and functional change.

d.
Consistent use of ranges of repetitious identical parts will encourage controlled performance testing, and allow the use of factory production techniques.

e.
The location and integration of mechanical engineering services is especially important, and a technique of co-ordinating dimensions and defining routes and locations is particularly necessary in order to relate these services to the basic anatomy of the building.

f.
Initial internal planning and future alterations are more easily carried out because of the orderly nature of the grid meshwork.

g.
The general content of drawings can be simplified and time saved in the preparation of working drawings.

The grid pattern proposed for Loughborough is a mesh of four grid networks defined as follows:

1. Master Grid:
A network of gridlines defining the structural space unit of growth: the gridline having a thickness determined by the need for vertical circulation space.

2. Planning Grid:
A network of gridlines defining the smallest increment of space (area) common to all predictable sizes of rooms and spaces: the gridline having a thickness determined by the thickness of partitions and external walls.

3. Structure Grid:
A network of gridlines defining or defined by the horizontal structural elements: the gridlines having a thickness determined by the thickness of structural members.

4. Services Grid:
A network of gridlines defining the main paths of service runs. At Loughborough this is less a grid than a space left over between structural grids, coincident (but in another dimension) with the planning grid to allow services to avoid structural elements and utilise hollow partitions for services to rooms. Primary and secondary vertical risers can occur anywhere. However, in practice the main vertical risers are usually best placed close to the more permanent features of the plan, such as the stairs and lavatories. Main vertical risers do not have to be in the link strip (i.e. the strip of structure spanning the thickness of the master grid), but this is often the most logical location. Any system of servicing which defines specific spaces for vertical risers is obviously more limiting than a system which allows complete freedom.
In the Loughborough solution there is complete freedom for vertical risers, but we expect some formal patterns to emerge as and when different types of laboratories are required. The advantage is that these patterns will not be permanent.

These four grid networks are meshed together to form a tartan of gridlines throughout the development.

The basic module for the planning grid is 3 ft. 9 in., the planning gridline thickness being 9 in., giving a 9 in. x 3 ft. x 9 in. dimensional pattern. The planning grid is the generator of all the grid networks, and the structure, services and master grids are, therefore, multiples of the basic module.

The dimension of 3 ft. 9 in. was chosen because it satisfied the following requirements:

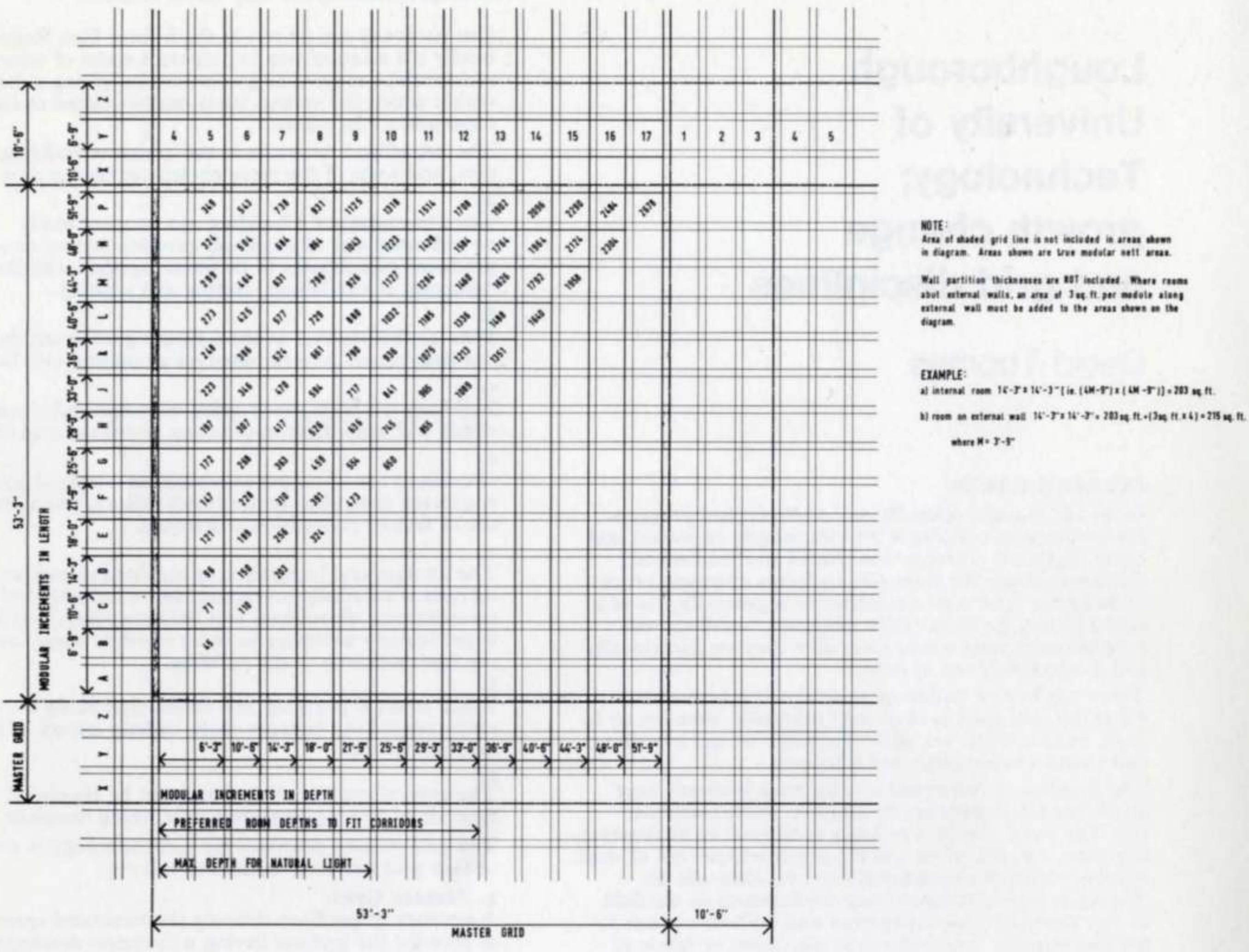


Fig. 1
Diagram showing range of modular room areas.

a.
The gridline thickness of 9 in. allows fairface brickwork to be used in partitions, either as double skin (i.e. 9 in.) or single skin ($4\frac{1}{2}$ in.) construction. Although the validity of the adaptable approach is based on the use of easily demountable factory produced partitions, there was a need for some more *permanent* sound-proof partition, particularly in the heavy engineering laboratories. It was necessary, therefore, to take into account the probable use of brickwork in some areas.

To achieve a reasonable sound reduction through a demountable partition system, where the weight factor cannot be utilised, a double skin design with a cavity is required, and an overall thickness of 9 in. allows this principle to be used.

b.
The distance apart of these 9 in. gridlines—the 3 ft. dimension—is a multiple of the gridline thickness. This was essential, bearing in mind brick dimensions, but also the resulting 3 ft. 9 in. basic module proved to give:

i. a basic modular area, multiples of which gave the greatest choice of room sizes approximating to those which could be most reasonably predicted.
ii. a basic module, multiples of which allow reasonable circulation widths (i.e. stairs and corridors).
iii. a basic module which would accommodate laboratory furniture layouts.
iv. a basic module which would allow a wide range of door opening sizes in either a panel or brickwork partition system.
v. a basic module which would give optimum positions for light fittings and the spacing of electrical and heating outlets.
vi. a basic module, multiples of which gave the optimum dimensions for the master grid network.

The master grid dimensions were determined by dimensional requirements for (a) the internal planning of double banked single corridor buildings requiring natural daylighting from both sides, (b) likely optimum sizes of laboratories, lecture theatres, seminar rooms, etc. (c) the optimum sizes of external courtyards in relation to the heights of surrounding buildings and (d) the optimum sizes of vertical circulation routes.

In order to have a central corridor of two modules width (i.e. 6 ft. 9 in. clear) in a double banked plan, it is essential that the master grid dimension between gridlines (i.e. the dimension of the narrowest possible building) should be an even number of planning modules *plus* one gridline thickness. This gives a 9 in. planning gridline at the perimeters for external walls where necessary.

The Loughborough master grid dimensions are:

gridline thickness	= (3M-9 in.)
dimension between gridlines	= (14M+9 in.)

where M = 3 ft. 9 in.

This gives actual dimensions of 10 ft. 6 in. and 53 ft. 3 in. Thus, the narrowest possible building is 53 ft. 3 in. wide. With a single central corridor of 2M-9 in. width, this produces room depths on either side of the corridor of 22 ft. 6 in., which for a floor to ceiling height of 10 ft. allows natural daylighting for the full depth.

Other factors which determine the dimensions of the master grid are:

a.
The size of the structural unit of growth. The Loughborough structural unit of growth is a square 53 ft. 3 in. x 53 ft. 3in. supported at the four corners. The width of the master gridline (10 ft. 6 in.) is covered by a simple one-way spanning structure supported on adjacent 53 ft. 3 in. squares. It is in this link zone that the vertical circulation (lifts and stairs) is *always* located.

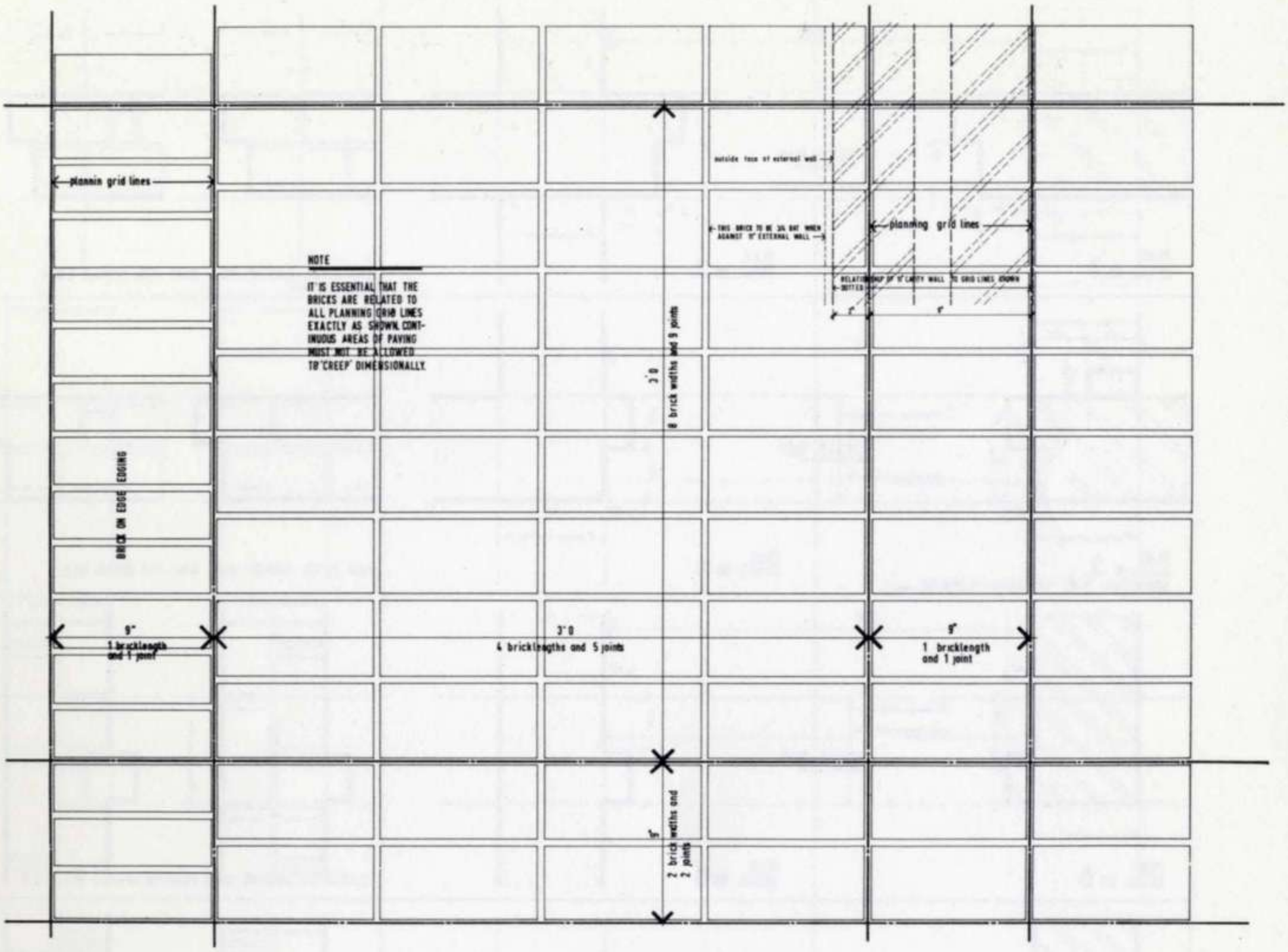


Fig. 2
Typical standard detail working drawing showing layout of brick paving in relation to planning grid.

The decision to provide a 53 ft. 3 in. x 53 ft. 3 in. two-way spanning structure was influenced by the need for adaptability. Clearly, the largest possible spaces free of vertical structure are desirable for ease of planning. To be economical, the resulting long spans require a deep girder solution which, given a load bearing ceiling, allow what is virtually a services floor below each normal floor. The advantages of this system are obvious since all services can be run to any part of the plan with complete freedom.

Vertical service runs can occur at any convenient point in the plan and, if within these structural space units a Musgrove-type laboratory plan and vertical service system is required, it can easily be accommodated. Unlike the Musgrove solution, however, it has the added advantage that at some future date a radically different system can be adopted, since the ducts are never structural or necessarily associated with structure.

b.

The site circulation system. The master grid dimensions must allow roads of reasonable widths to be planned between and under buildings, and allow economical car parking layouts.

c.

The possible rate of growth may determine the size of the structural unit of growth in that too large a unit may be economically unsuitable. In some ways a small structural unit of growth might be more acceptable since it would give greater flexibility in the choice of the amount of growth. However, this consideration must be weighed against the advantages of column-free spaces and the services void. The Loughborough solution has proved very economical, probably because of the logic of the constructional system and the fact that the span/depth ratio is correct. The structure is in fact very light, weighing no more than a 53 ft. 3 in. square 9 in. deep slab.

Having fixed the planning grid dimension and, therefore, the master grid dimensions it followed that the structure grid would also be 3 ft. + 9 in.; but not coincident with the planning grid. This was essential to avoid structure clashing with either services rising vertically into hollow partitions or columns interfering with the partitions on the planning grid.

These decisions relating to the dimensions of the grid networks were made against the background of the philosophy of the Master Plan, and were proposed after some research into the specific problem of Loughborough, and also on a pragmatic attitude based on experience of designing various types of buildings from research laboratories to hospitals.

CIVIL ENGINEERING BUILDING

The real test of the proposals came in applying them to a specific problem—the first building within the framework of the Master Plan—the Civil Engineering building.

The experience of designing this building against the background of the dimensional discipline has been interesting, and so far the original Master Plan proposals have been fully justified although inevitably certain modifications have been necessary.

The Civil Engineering building is in two parts—the Teaching Building, consisting of lecturers' rooms, seminar rooms, drawing offices and lecture theatres, and the Laboratory Building, consisting of 'heavy' laboratories such as workshops, Concrete Technology, Hydraulics, Models, etc.

Perhaps the most significant variation from the original proposals has been in the structure.

The structure proposed for academic buildings in the Master Plan Report was a two-way post-tensioned concrete girder system. This was subsequently designed in great detail and put out to tender. As a result of rather high tenders the structural method was considerably revised in

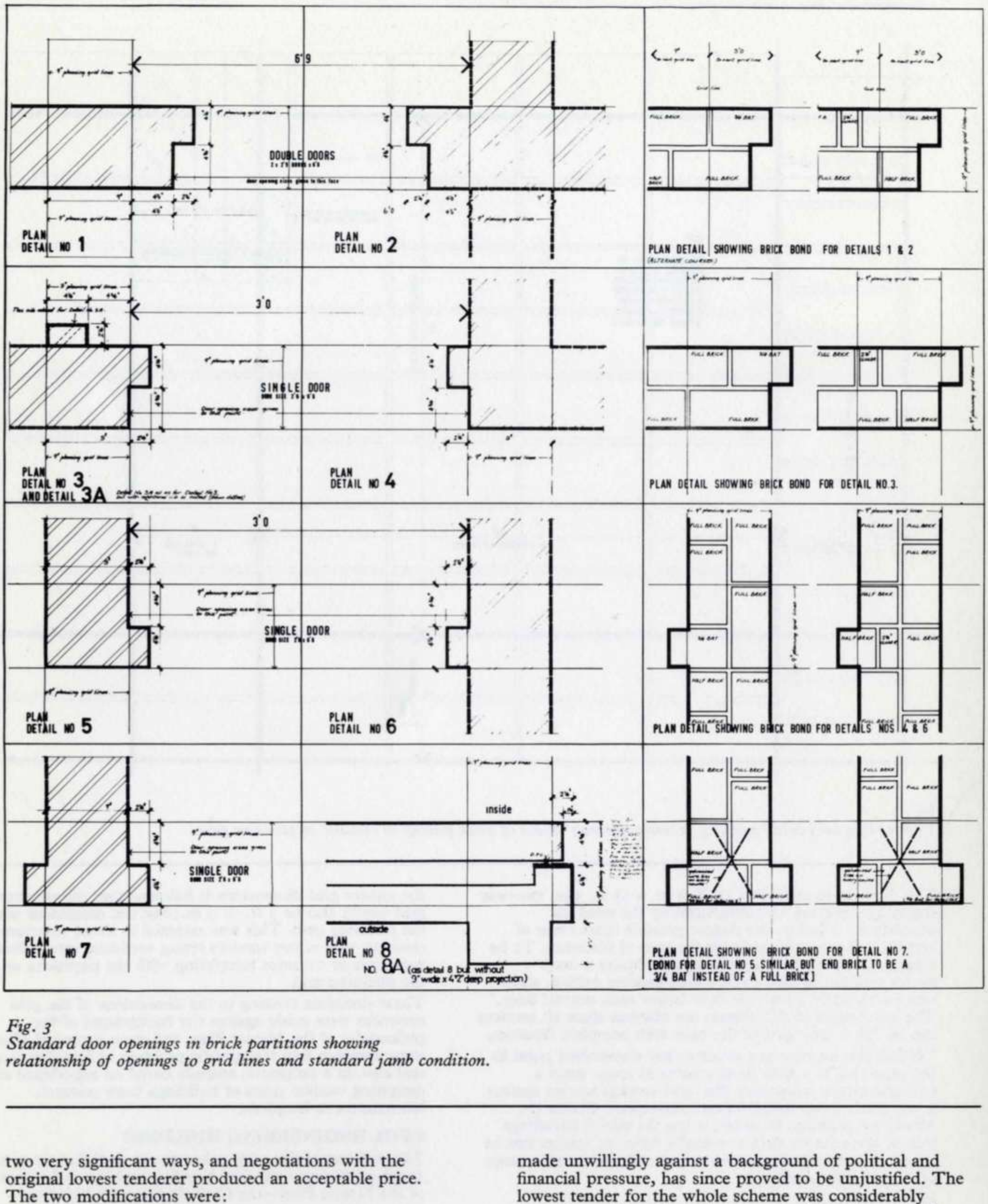


Fig. 3
Standard door openings in brick partitions showing relationship of openings to grid lines and standard jamb condition.

two very significant ways, and negotiations with the original lowest tenderer produced an acceptable price. The two modifications were:

a.
For the Teaching building a precast concrete girder system was proposed spanning one way into an edge girder which, by means of in situ joints with the columns, became a portal frame. This revision eliminated both the costly joints between girders and the post-tensioning necessary in the two-way spanning scheme.

b.
For the Laboratory building, where an overhead travelling crane dictated a high single storey linear building, intermediate columns were introduced eliminating the edge girder portal frame. These columns are used to support the crane rails, a mezzanine floor and the roof girders. The servicing problem is necessarily at ground level and there is, therefore, no services void, resulting in no ceiling. The roof deck is a lightweight metal tray system.

The decision to compromise on this structure, which was made unwillingly against a background of political and financial pressure, has since proved to be unjustified. The lowest tender for the whole scheme was considerably lower than either the estimate or the U.G.C. cost limit, and it is clear that the structure for the Laboratory building could have been identical to the Teaching building structure, while still keeping the total building cost below the estimate and U.G.C. figures.

It should be noted, however, that this compromise in the Laboratory building affected only the structural system. The 53 ft. 3 in. x 10 ft. 6 in. master grid dimensions have been rigidly observed, and have proved completely satisfactory in practice, and there seems to be no need to modify these dimensions in the light of our experience in the Civil Engineering buildings.

Planning grid

Similarly, after producing many different sketch designs for a constantly changing client's brief, we have found the planning grid dimensions satisfactory. The philosophy of designing for growth and change by producing the maximum possible freedom of choice has proved itself invaluable.

During the design stage of the first building the client's brief changed frequently and drastically, and we have been able to plan within the grid discipline the full range of university academic accommodation.

Partition system

To allow for rapid change and growth it was necessary to develop a partition system to the following performance specification:

1.
The partition system must be modular, i.e. a panel or post and panel system related to 3 ft. 9 in.

2.
Each panel of the partition system must be capable of being removed individually (i.e. unlike many systems it should not be necessary to take down a whole wall in order to remove a central panel to insert, say, a door).

3.
The panels should be such that a wide variety of finishes can be applied, and glazed panels should be available.

4.
An accessory fixing system should be incorporated in such a way that panels are not damaged and no making good is necessary. This fixing system must be capable of carrying loads of say 100 lb. cantilevered 2 ft. from the partition.

5.
The partition system must allow for deflection in the structure.

6.
The partition system must not have permanent fixings either to the floor or the ceiling.

7.
The partition system must give a good sound insulation when necessary—at least equivalent to a $4\frac{1}{2}$ in. brick wall plastered both sides.

8.
The partition must be easy to assemble and reassemble.

9.
The partition construction must satisfy fire requirements for means of escape.

The partition system which met this specification was designed in close collaboration with Tenon Contracts Ltd., and the eight points of the performance specification were met in the following ways:

1.
Initially it was thought that there would be just two panel sizes for the 3 ft. + 9 in. grid—a 3 ft. panel and a 9 in. panel. However, it was obvious that this would involve an uneconomical numbers of posts and uneconomical widths of panel. Finally, panels with format dimensions of 9 in, 3 ft. and 3 ft. 9 in. were chosen; the 3 ft. 9 in. panel being the most common with the 3 ft. and 9 in. panels as make-ups where necessary.

The format dimensions principle is important in developing a modular system. The format dimensions of a brick are 9 in. x $4\frac{1}{2}$ in. x 3 in., whilst the actual dimensions are $8\frac{5}{8}$ in. x $4\frac{1}{8}$ in. x $2\frac{5}{8}$ in., allowing a constant $\frac{3}{8}$ in. joint all round. The relationship between the width, depth and height dimensions allows the brick unit to be fitted together in a variety of ways, maintaining a regular joint thickness. Similarly, in designing a panel for the 3 ft. 9 in. planning module it was necessary to treat the 3 ft. 9 in. as a format dimension, and the post and junction as a joint which bears a constant relationship to the gridlines. *This is most important.* The actual panel size being 3 ft. $7\frac{3}{4}$ in. for the 3 ft. 9 in. format dimension, 3 ft. 1 in. panel for the 3 ft. format dimension, and $5\frac{1}{2}$ in. for the 9 in. format dimension. If this principle is not observed many different panel sizes will result. Even using this principle we have four panel sizes at Loughborough, (3 ft. $7\frac{3}{4}$ in., 3 ft. 1 in., $5\frac{1}{2}$ in. and a 2 ft. $7\frac{1}{4}$ in. make-up panel) but before the post/grid relationship was defined the number of panels was much greater.

2.
The panels are held in position by removable cover strips which allow each panel to be removed easily.

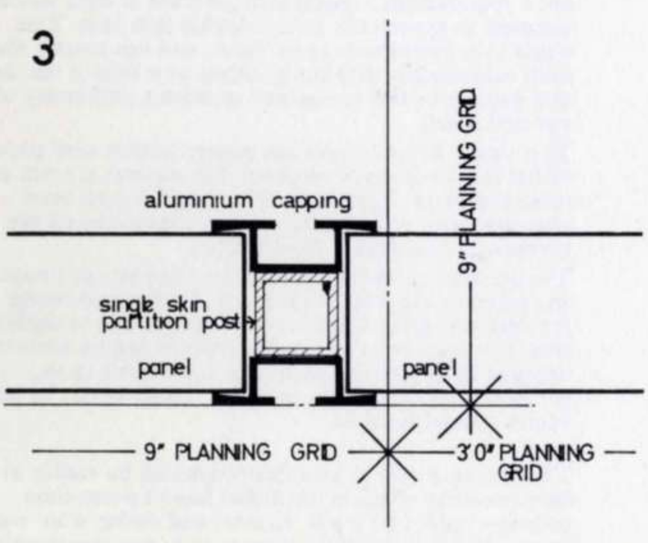
3.
The panels are constructed of two $\frac{3}{8}$ in. thick plasterboard panels bonded together and faced both sides with $\frac{1}{8}$ in. O-Board. The O-Board can be finished with veneers, laminates, p.v.c. sheet, vinyl or paint.
Tenon's process of manufacture of panels requires the edges of bonded sheets to be trued off after bonding. This means that a 4 ft. standard panel must always be shot. 3 ft. 9 in., therefore, is not considered an uneconomical finished size, although there is a degree of wasted material.

4.
The panel retaining cover strips are designed in anodised aluminium to accept a bolt fixing which allows many accessories to be easily fixed in any vertical position in the posts of the partition system.

Tenon Contracts Ltd. have produced a wide range of accessories which include:

a. Cantilever brackets of varying length (all in anodised aluminium)
b. Coat hooks (all anodised aluminium)
c. Crash rails and brackets (timber and anodised aluminium)
d. Blackboards
e. Notice boards
f. Cantilever cupboard units (produced in conjuction with Conran Ltd.)
g. Various fixings brackets.

3



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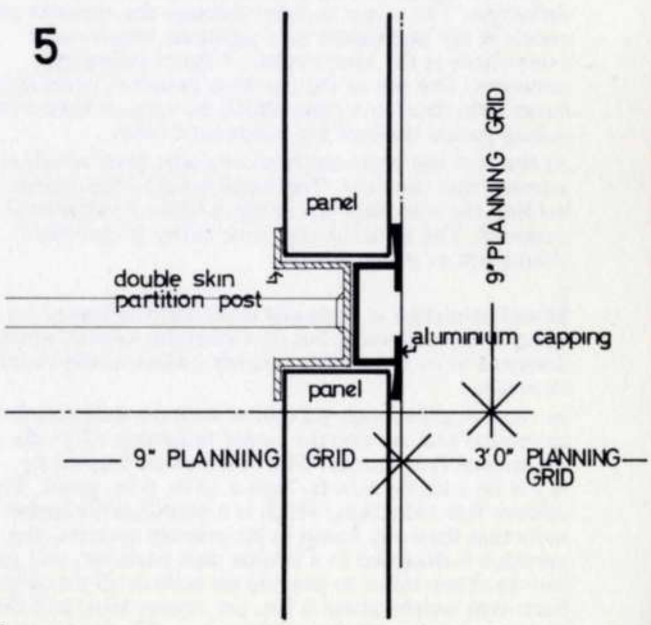


Fig. 4
Partition post details. (a) single skin partition (b) double skin partition.

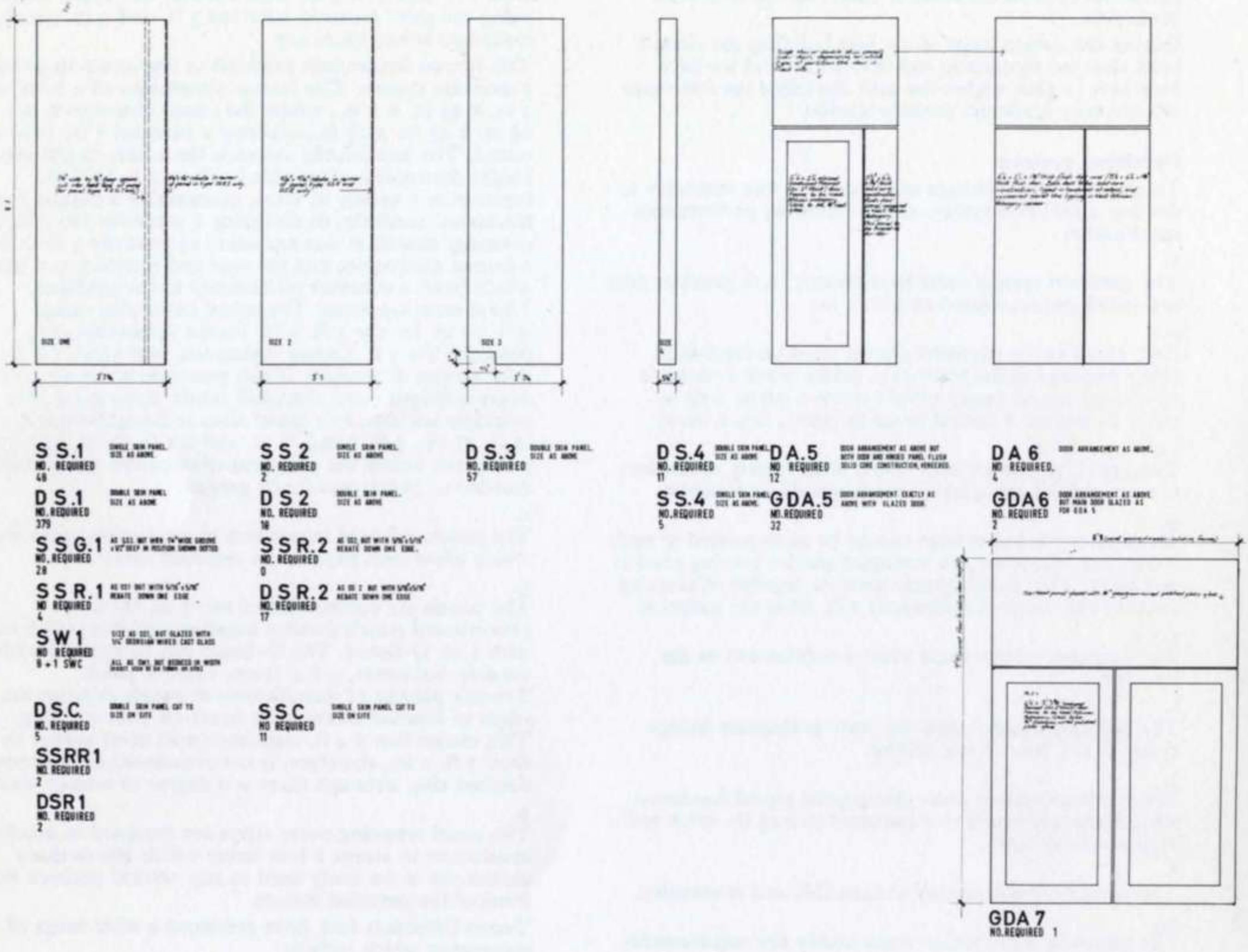


Fig. 5
Working drawing showing schedule of partition panel types.

The performance specification requirement for loading has been achieved by using steel posts within the partition thickness to which the anodised aluminium cover strip is screwed.

5. & 6.
The partition posts are held at the head by a spigot fixed to the ceiling, providing a sliding joint which allows deflection. The spigot is fixed through the concrete ceiling panels at the permanent hole positions which occur everywhere at the intersection of the planning grid networks. The top of the partition panels is covered by a cover strip fixed to a plate which, in turn, is fixed to the ceiling panels through the permanent holes.

At the foot the posts are fixed to a sole plate which is screwed into the floor. The small holes of the screws holding the sole plate are plugged when a partition is removed. The partition can have either an anodised aluminium or p.v.c. skirting.

7.
Sound insulation is normally most easily achieved by using heavy materials, but in a partition system which is designed to be removable weighty panels would be an anomaly.

In the Loughborough partitions we have achieved, in a controlled test, an average sound reduction of 47 db. over the normal frequencies. This test was performed by A.I.R.O. Ltd. on a 10 ft. high x 10 ft. 6 in. panel. To achieve this reduction, which is a considerably higher reduction than any found in proprietary systems, the partition is designed as a double skin partition, and great care has been taken to provide air seals at all junctions. Each skin weighs about 8 lbs. per square foot, and the cavity contains a blanket of fibreglass. The posts are constructed of pressed steel U-sections jointed together (for strength and stability) by cross pieces forming a lattice member. The cavity in the partition is used for services when necessary.

The double skin partition with vinyl covering costs 24/- a square foot, depending on the number of doors. In order to reduce the total cost of the partitions the double skin is only used where a sound reduction problem exists between spaces. In other cases where sound reduction is not a requirement a single skin partition is used which is identical in appearence to the double skin unit. This single skin partition is $1\frac{1}{2}$ in. thick, and has exactly the same relationship with the gridlines as a skin of the double skin partition—this is essential to ensure uniformity of size and detail.

This single skin partition has square section steel posts to which the cover strips are fixed. No services are run in these partitions. The single skin partition with vinyl covering costs 13/- per square foot, depending on the number of door panels incorporated.

The use of single skin partitions has a further advantage. If all partitions are 9 in. thick a considerable percentage of the enclosed space within the building is lost as usable area. If, therefore, a reasonable balance can be achieved between single ($1\frac{1}{2}$ in.) partitions and double (9 in.) partitions the area lost in partition thickness can be kept within normal bounds.

8.
The planning grid at Loughborough will be visible in the floor covering which is vinyl tiles laid in a two-tone pattern—light 3 ft. x 3 ft. squares and darker 9 in. wide strips. The dark colour is used to hide any discoloration which may be evident when partitions are removed from a 9 in. strip.

Similarly, the grid network is delineated in the concrete ceiling panels, the 9 in. grid strip downstands forming a shallow coffer pattern of 3 ft. x 3 ft. recessed squares.

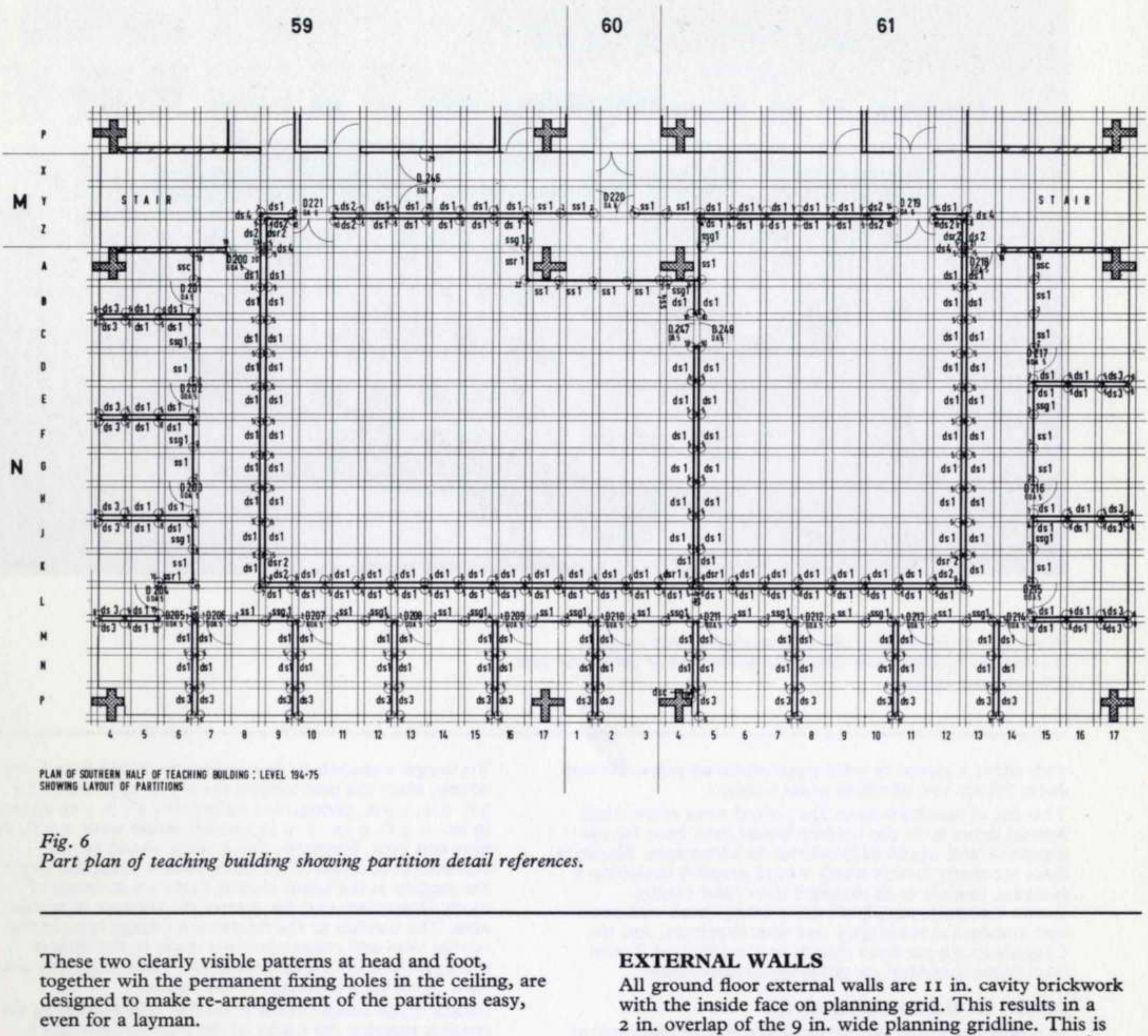


Fig. 6
Part plan of teaching building showing partition detail references.

These two clearly visible patterns at head and foot, together wih the permanent fixing holes in the ceiling, are designed to make re-arrangement of the partitions easy, even for a layman.

9.
Fire resistance is obtained by the O-Board which gives an O class resistance to flame.

CEILING

The ceiling at Loughborough forms the floor of the services void within the 5 ft. deep structure. Since the partitions finish at ceiling level it is important that the ceiling provides as good sound reduction as the best partition. To satisfy the loading conditions of the services void, and to give this sound reduction, the ceiling is made up of reinforced concrete panels supported on the bottom booms of the floor girders. The planning grid pattern is cast into the soffit of the ceiling panel, forming a shallow coffer pattern. At the intersection of each 9 in. wide x 1 in. deep rib a permanent 6 in. × 6 in. hole is formed. This hole allows fixing positions throughout a 53 ft. 3 in. x 53 ft. 3 in. square at 3 ft. 9 in. centres in both directions for—

a. The warm air heating outlets
b. Light fittings
c. Partitions
d. Service drops inside double skin partitions
e. Vertical service risers of small dimensions.

When these holes are not being used for any of the above purposes they are blanked off with metal cover plates.

The ceiling panels will have a fair-face finish and are to be painted white. Where sound adsorption is required, 3 ft. x 3 ft. acoustic tiles will be inserted in the coffer.

EXTERNAL WALLS

All ground floor external walls are 11 in. cavity brickwork with the inside face on planning grid. This results in a 2 in. overlap of the 9 in. wide planning gridline. This is unimportant since it always occurs at the perimeter of a building.

At first floor level and above, the external wall consists of precast concrete cladding to the 5 ft. deep edge girders. From floor to ceiling a glazing system has been developed with Messrs. British Challenge Glazing Co. Ltd. which has black anodised aluminium glazing bars of a more sophisticated patent glazing type, coinciding with the planning grid modules—3 ft. x 9 in. x 3 ft., etc. etc. This observance of the planning grid is essential in order to join either double or single skin partitions to the external wall.

Grey glass is used throughout, reducing glare and to some extent heat gain, and also exposing less of the interior than would be the case with ordinary glass.

Natural ventilation is by top hung outward opening windows set in the 3 ft. module at 5 ft. above finished floor level. From first floor upwards the whole external wall, including the girder precast concrete cladding panels, is designed to be removable when expansion is required by the addition of 53 ft. 3 in. squares.

DOORS

Throughout this first building standard doors have been used—2 ft. 6 in. x 6 ft. 6 in. and 2 ft. 9 in. x 6 ft. 6 in. In the brickwork partitions a standard door frame lining and brickwork jamb detail has been used throughout, with two constant relationships to the planning grid, one for single and one for double doors.

In the demountable partition system the standard doors are set in a standard partition panel width (3 ft. 9 in.)

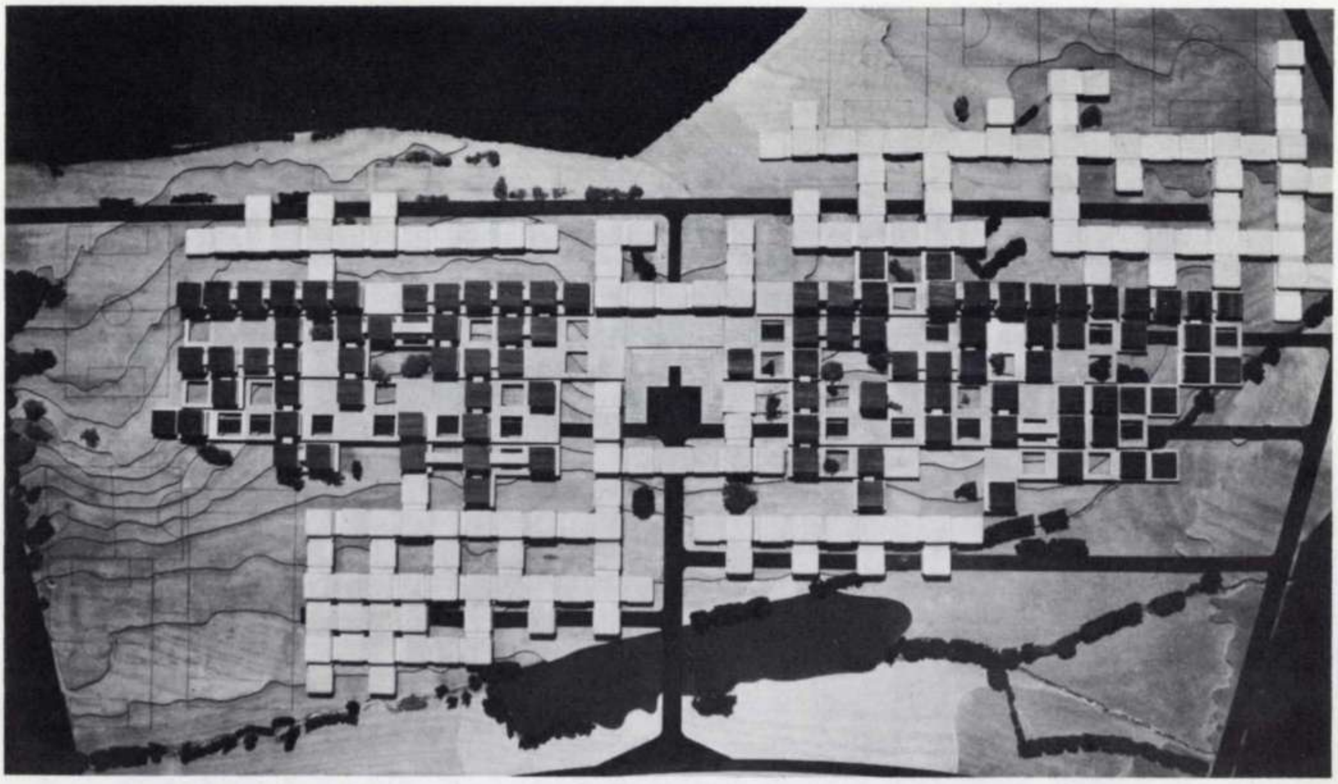


Fig. 7
Plan view of the model of the University for 5000 students showing the building units within the framework of the master grid. (Photo: John Donat)

with either a glazed or solid panel make-up piece. Double doors fit into two partition panel widths.

This use of standard doors has proved most economical. Special doors to fit the module would have been far too expensive and would have offered no advantages. Similarly there are many factors which would prohibit designing a planning module to fit standard door sizes exactly. Tenon Contracts supply all doors ready fitted with their own standard ironmongery and door furniture, and the Loughborough partition system uses a standard Tenon door frame extrusion complete with rubber seals.

BASIC BUILDING SERVICES

In a building where the declared aim is to allow constant quick and easy rearrangements of the internal planning, it is essential that the heating system is designed to be either equally rearrangeable or constantly correct whatever plan rearrangements take place.

The latter proposition is virtually impossible with an economical method of heating, and so the heating system has been designed to allow rearrangement. Heating is by means of warm air introduced through diffusers which plug into the 6 in. x 6 in. permanent ceiling holes. These diffusers are connected by flexible ducting to a heater battery and fan, housed in each of the 53 ft. 3 in. square service voids. The source of heat is a central gas-fired boiler plant. The flexible ducting and plug-in diffusers allow the heating to be directed where required. The system can also be used for ventilation in summer.

ELECTRICS

Lighting is either by means of tungsten fittings which again plug into the 6 in. x 6 in. permanent holes or by fluorescent tube fittings. The fluorescent tube fittings are 4 ft. 6 in. long using a 4 ft. tube, and always relate to a 9 in. x 3 ft. x 9 in. grid dimension.

The use of 4 ft. tubes, whilst relating to the modular dimensions and giving a wide range of flexibility in light output (by varying the number of tubes in a fitting for instance), is not as economical, in terms of running costs and lamp replacement costs, as using 5 ft. tubes. We are in the process of considering changing over to 5 ft. tubes, and whilst this may not be as aesthetically acceptable as 4 ft. tubes, from the point of view of fitting the module, there is no really strong argument against this.

To design a module to fit a light tube would have been wrong, since the best module for a 5 ft. tube would be 3 ft. 9 in. + 9 in. giving, over ceiling ribs, a 5 ft. 3 in. dimension (9 in. + 3 ft. 9 in. + 9 in.) which would allow a 5 ft. tube plus end caps. However, this module would be uneconomical in terms of partition panels. Also, the larger the module is the fewer choices there are in terms of room dimensions and the greater the wastage of usable area. The conduit to the fluorescent fittings runs in the service void and connections are made to the fittings through the 6 in. x 6 in. permanent holes which are also used for fixing the fittings to the ceiling.

Switch drops *always* occur in double skin partitions, the conduit entering the cavity of the partition through a 6 in. x 6 in. ceiling hole and feeding switches on a 3 ft. 9 in. wide aluminium plate fixed to the partition 3 ft. above finished floor level.

This plate is designed to take socket outlets, telephone outlets, T.V. outlets, etc. as well as switches.

PLUMBING AND DRAINAGE

Just as the ceiling has a pattern of permanent 6 in. x 6 in. holes at the intersection of the gridlines, so the floor has 6 in. x 6 in. expanded polystyrene squares cast in at the intersection of gridlines. These squares can be easily knocked out when plumbing and drainage pipes are required to go through the floor. If the pipes are later removed the floor slab and the tile floor finish can easily be made good.

Normal domestic plumbing and drainage will be within the cavity of a double skin partition, a standard detail having been designed for cantilever w.c.'s and basins to fit the partition system.

Laboratory plumbing and drainage will normally feed directly from benches into the services void below.

LABORATORY FURNITURE

Laboratory furniture is designed as a bench system with removable under-bench units. Standard lengths of worktop of modular dimensions are used, and the under-bench units are in two widths, 3 ft. and 1 ft. 6 in. Leg frames are at 3 ft. 9 in. centres. The width of benches is most often 2 ft. 6 in. front to back, giving (10 ft. 6 in. − 5 ft.) = 5 ft. 6 in. clear between benches for both single laboratories (benches down both sides) and laboratories with peninsular or island benches.

GENERAL CONCLUSIONS

Having completed the designs and working drawings of the first building for Loughborough certain assessments can be made of the method of working to a grid discipline.

1.
There must be an absolutely ruthless observance of the grid pattern. Any divergence from this discipline quickly causes widespread chaos. Observing the discipline must become second nature to all members of the design group.

2.
The actual dimensions of the grid network are relatively unimportant—it is the principle that counts, but we have so far found no reason to vary the basic dimensions of 3 ft. and 9 in. Certainly, we would not recommend smaller dimensions which would increase the cost of the partitioning and glazing systems (cost being directly proportional to the number of posts), affect the economic distribution of services holes, and perhaps be wasteful in planning possibilities.

The 3 ft. and 9 in. module allows staircases to fit both in width and length, allows double module or triple module corridors and even single module corridors. A single module corridor with double skin partitions would be 3 ft. wide and by using single skin partitions on each side this width can be increased to 4 ft. 3 in.

We have found that a module based on 3 in. suits—

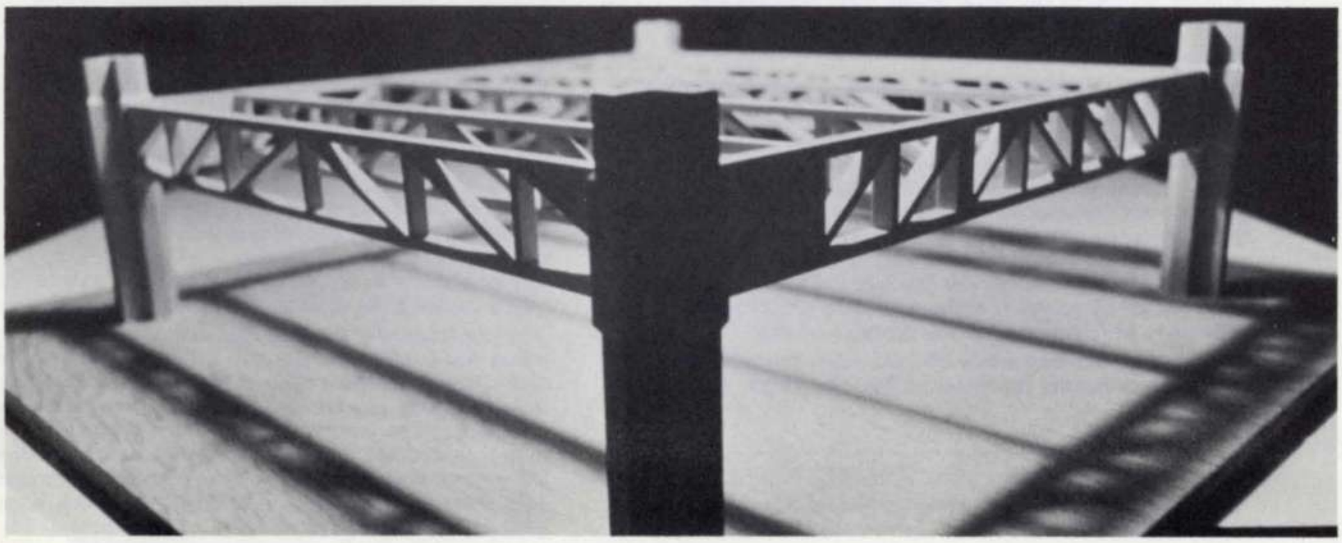
a. brickwork (the 4 in. module does not work for brickwork)

b. all floor tiles based on 3 in. i.e. 6 in. x 6 in., 9 in. x 9 in. 12 in. x 12 in., p.v.c., lino and vinyl tiles, and 4½ in. x 4½ in. quarries, brick paviors, etc.

3.
It is important to produce performance specifications for elements, and then to work with the manufacturers of each element in producing a design and prototype to meet the specification, within the planned cost.

Fig. 8
General view of the Civil Engineering Teaching building.
(Photo: David Weaver)

Fig. 9 below
General view of structural unit of growth—53 ft. 3 in. x 53 ft. 3 in. square.
(Photo David Weaver)



Large concrete pours especially on the Barbican Scheme

Roger Findlay

In civil engineering work, dams for example, the pouring of large volumes of concrete is quite common and the contractor is well equipped for its handling. His operations are planned in advance and are as fully mechanized as possible. However, this type of work may be outside the normal experience of some building contractors. On the Barbican Redevelopment Scheme the foundation loads are high and this has led to many pile caps being of quite substantial size, requiring concrete pours of several hundred cubic yards at one time. The methods that were adopted are applicable to the average building site and it has been thought that the experience gained on these pours may be of some benefit to others. Notes on general concreting procedure are already available, for resident engineers, in the library(1)*

The adjective large need not necessarily apply to volume. It was found that as much care had to be taken with the formwork of a cap containing 13 cubic yards but 4 ft. thick, as one containing 300 cubic yards.

RATES OF CONCRETING

The most important decisions the contractor makes are those involving the speed of concreting because all other decisions, source of supply, transportation, size of gang, the vibration capacity, formwork design, depend on this. The speed with which the job is done will depend on the slowest operation. On compact sites, where the concrete can be handled direct from the mixer to the shutter by crane and skip, the output of the mixer will probably be the controlling operation. On larger sites, where one crane will not cover the whole area, it will normally be transportation which will govern the rate of concreting.

SOURCE OF SUPPLY

Generally, the concrete will be supplied by the site mixer for economic reasons. If the site is confined and there is no space for a mixer, or the output of the mixer means that the pour will run on into overtime and become uneconomic, ready-mixed concrete may be used.

If the contractor intends to use ready-mixed concrete, he must consider traffic conditions when planning his work, particularly if it is the only source of supply. In London it is prudent to try to plan concreting outside peak traffic periods. On Phases 2 and 3 of the Barbican Redevelopment Scheme several of the larger pours were planned for Saturday mornings and they were always successful.

TRANSPORT

Concrete may be transported around the site in a number of ways. The methods used on the Barbican Scheme were by crane and skip for the pours adjacent to the mixer but as the sites cover a large area it was generally by dumper or truck, either the contractors' own, or ready-mix. Concrete pumps were considered but were rejected because most of the concrete on the site was going into the normal small pours and there would be a very large wastage in the pipeline and it was not worth the trouble and cost of installing them for the large pours alone. Concrete placers and conveyors were rejected on the grounds of cost.

One contractor relied solely on half-cubic yard dumpers and crane to handle the concrete into the forms. His rate of concreting was usually around 15 cubic yards per hour with a mixer capable of producing 27 cubic yards per hour. This meant that a large pour went on for a very long time but at the same time he could supply concrete from the mixer to other parts of the site. The dumpers travelling over the rutted ground did cause a little segregation but, provided adequate vibration was given, the cast concrete was satisfactory.

The other contractors used 3 cubic yard trucks and supplemented these with ready-mixed concrete for the main bulk of the pour and were achieving rates of over 50 cubic yards per hour. Again they balanced their ready-mixed concrete supply so that the site mixer was available for supplying concrete to other parts of the site. Most of the large pours were below ground-level and so the large volumes of concrete were most conveniently handled on chutes into the forms, using a crane and skip only in the areas inaccessible to lorries and for finishing off. No trouble occurred due to segregation on the chutes, which were roughly at 45°, despite some of their being 40 ft. long on Phase 4. On one pour very close to the road, the chutes had to be about 60° to the horizontal and there was segregation due to large aggregate rolling down first. In retrospect, the obvious method would have been to chute the concrete from the truck to an elephant's trunk.

PLACING AND COMPACTION

The placing and compaction of the concrete is the most important operation for the success of the pour and it is perhaps the simplest to carry out, so long as the contractor has a method and maintains strict control. He should have sufficient men on the job to see that the concrete does not pile up into heaps, otherwise this will lead to segregation because the large aggregate will roll down the sides or large volumes of concrete will remain concealed and unvibrated underneath. It was found by all the contractors on the Barbican Scheme that optimum results were obtained by using one 3 in. vibrator poker for every 6 to 7 cubic yards of concrete poured every hour.

As a result of using chutes at various places around the sides of the pour, which is essential for high rates for pouring, it meant that concreting was carried out in layers rather than in a series of vertical faces moving across the cap. The trouble with concreting in extensive layers, necessary for larger pours, is that if a hold-up occurs in the concrete supply or the concrete is hardening or losing workability quickly, some concrete may not be integrated with the next layer before initial set is achieved. This actually happened on two occasions, resulting in the contractors concerned having to break out a total of 400 cubic yards of concrete. If concreting is being carried out on a face, there is a chance to get in a stop-end and make a construction joint if there is trouble.

WORKABILITY

Apart from the general problem of having concrete with the right consistency and workability for handling and for working around reinforcement, there is the additional problem of keeping the concrete plastic long enough to receive the succeeding layer. The Barbican experience was that loss of water through evaporation was the cause of most of the trouble. The worst places for this to happen were either in the drums of the ready-mix trucks or where the pour is in an excavation forming a sun-trap. On both occasions when things went wrong retarders had not been used. On one occasion the ready-mix trucks were held up in the heavy traffic in the City streets. On the other there was a combination of reasons, ready-mix trucks in the summer heat, 6,000 psi concrete and the pour was in an excavation forming a wonderful sun-trap.

The conclusions were that it was always wise to use a retarder with ready-mixed concrete or the high grades of concrete, otherwise it was not necessarily important but should be seriously considered beforehand. The action of the retarder was to delay the chemical action and thus reduce the rise in temperature due to the heat of hydration which meant a far smaller loss of water through evaporation.

The mix normally used on Barbican Phase 4 for these pours was a 4,800 psi mix and water/cement ratio of 0.43 and aggregate/cement ratio of 6:1 using $1\frac{1}{2}$ in. aggregate maximum size sea-dredged aggregates and a retarding plasticizer. The contractors on Phases 2 and 3 had similar mixes, sometimes with, sometimes without, retarders. They were normally easy to handle without segregation and, apart from the two exceptions mentioned, were sufficiently workable at all times in the forms.

**Editorial Note: These are being revised by Alan Steele and the new edition should be ready by the end of the year.*

Fig. 1 right
Concreting pile cap with crane and skip.

Fig. 2 below
General view of Block 14,
Barbican Redevelopment, showing pile caps
in various stages of construction.

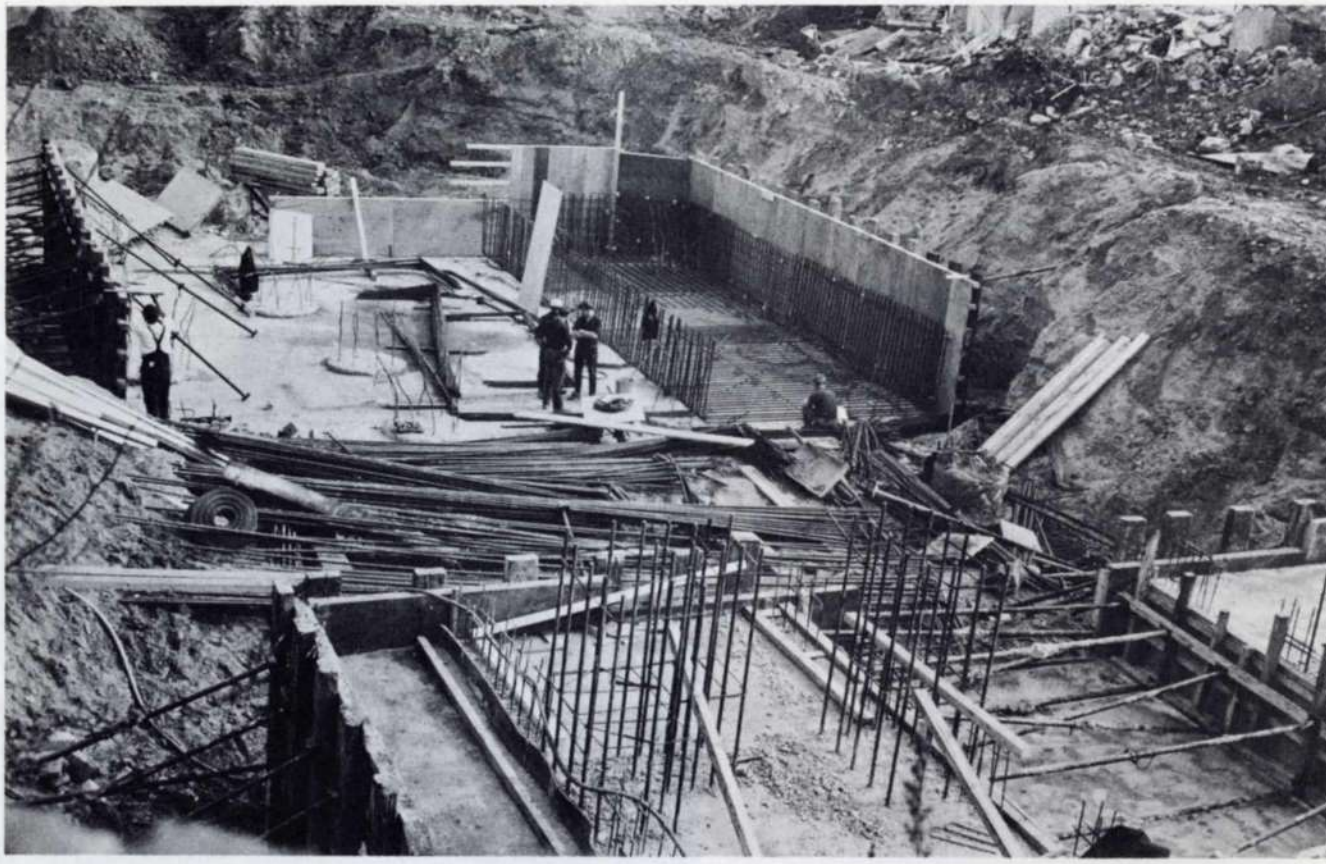


Fig. 3
300 cubic yard pour in course of preparation.
In the foreground a completed 300 cubic yard pour.

FORMWORK

The formwork for the large pours, even at low rates of pouring, is far more complicated than it first seems. The problems are, firstly, to stop the forms floating upwards and kicking out at the bottom, and secondly, to maintain the correct alignment. The latter is often not so important, because generally volumes of this size are associated with foundations and are not exposed in the finished work. On the Barbican Scheme the pile caps formed the sides of the subway for services. A warning for designers here—it is very difficult, and therefore very expensive, to get perfect alignment. What happens is you pay more money and you may still get a rough job. So avoid incorporating large pours into exposed work, especially if several sections have to be joined together.

Various schemes have been adopted on the Barbican Scheme for overcoming these problems. The chief precaution to take against formwork floating is to avoid too wet a mix. This, of course, has to be balanced against loss of workability. One early method on Phase 4 was to place timbers across the tops of the forms and hold the timbers down by wires to the reinforcement in the pile heads. Later a scheme was devised using old railway sleepers bolted to the backs of the forms at 2 ft. centres, which as well as adding weight, meant that they could be firmly braced. The forms were then strutted from the outside by driving lengths of heavy reinforcement through the blinding and right into the gravel underneath at 2 ft. centres and laying 12 in. x 12 in. timbers against them from which the forms could be propped using Acrow props at 2 ft. centres. The secret was not to do anything in half measures. Care was taken to avoid Acrow props at a steeper angle than 30° to the horizontal, because of the tendency to lift. Two ways of simplifying the strutting were where possible to strut from the sides of the excavation or to first construct alternate pile caps and then go back and strut the remaining pile caps from those cast first. With caps of irregular size it was found simplest to lay the blinding to the exact dimensions of the drawing and then use the thickness of the blinding as a kicker for the forms. The scheme adopted by the Phase 2 contractor was to set *Rawlbolts* into the blinding and secure them to the bottom shutter waling. The top of the shutter was tied against outward bursting with a scaffold ring beam.

LABOUR AND SUPERVISION

Once the source of supply and the method of transportation has been settled and the contractor has determined the vibration capacity based on the calculated rate of delivery, he can settle on the size of the gang. He must also arrange standby gangs for lunch breaks etc., not forgetting to include crane, mixer and truck drivers. In the Barbican area the unions must know what is planned. If the site is working to the civil engineering working rule agreement then there is no necessity to agree the overtime, associated with pours lasting 12 hours or more, with the unions. Under the building agreement this must be done, since for instance it has been known for a labour meeting to be called suddenly for after lunch and the success of a half-completed 300 cubic yard pour to be jeopardised.

Constant control is necessary and a responsible person must be present and controlling the operation at all times, not just on hand should trouble occur. This is very important because things can get out of hand very quickly when pouring at 50 cubic yards per hour. The resident engineer should also keep a constant watch on the progress of the work. From my experience there is no more important time than when the standby gang is working. They never seem to be men with much experience of concreting and vibration is always slapdash and there is no system of placing the concrete.

CONCRETE TEMPERATURE

Some measurements of temperature within large pours were made on Phase 2 in the early days. Lengths of plastic conduit or water barrel were cast into the pile caps to 1/3, 2/3 and 1/2 depth and filled with oil. The two caps involved were 60 cubic yards x 5 ft. thick and 100 cubic yards x 6 ft. thick. In both of them the maximum temperature was 100°F, recorded at half depth. As a result of these temperature checks it was not thought worthwhile using low heat cement which tends to be variable and unpredictable. The firm's *Cement Manual* gives some information on low heat cements, also mass concreting and hot weather concreting (2).

REFERENCES

(1)
Ove Arup & Partners Consulting Engineers. Site Supervision of Engineering Works, by K. J. C. Clayden. 1963.

(2)
Ove Arup & Partners Consulting Engineers. Cement Manual, by Turlogh O'Brien. 1966

Asbestos mine winder tower in Canada

K. J. C. Clayden

ASBESTOS

Asbestos is the only natural inorganic fibrous material to have attained commercial importance. It has been found in widely scattered areas around the world but deposits of commercial significance are not great in number. The extent of its use has grown substantially during recent years, particularly since the end of World War II.

Technology and industry apply the term asbestos to the fibrous crystalline varieties of five distinct natural silicates. Two main groups are recognized with respect to origin, mode of occurrence, mineralogical relationships, structure, composition and properties. They can be referred to as the serpentine asbestos group and the amphibole asbestos group.

VARIETIES

The ideal formulae for the five varieties of asbestos are:

Serpentine Asbestos	
Chrysotile	$Mg_3 [(OH)_4 Si_2 O_5]$
Amphibole Asbestos	
Crocidolite	$Na_2 MgFe_5 [(OH) Si_4 O_{11}]_2$
Amosite	$MgFe_6 [(OH)Si_4 O_{11}]_2$
Anthophyllite	$(MgFe)_7 [(OH)Si_4 O_{11}]_2$
Tremolite	$Ca_2 Mg_5 [(OH)Si_4 O_{11}]_2$

The larger deposits are found in veins in serpentine. Several theories have been put forward for its presence and the one which appears to find the greatest favour is that the crystals are formed by solution of the existing parent rock by water injected under high pressure through cracks in the rock and subsequent recrystallization within the cracks themselves. The fibres thus run perpendicular to the walls of the cracks and are parallel. The length of fibres varies from a fraction of an inch up to a normal maximum of about ¾ in. although lengths up to 3 in. are not unknown.

USES

Chrysotile, crocidolite and amosite are the principal forms of asbestos used for industrial purposes and each has its own application in commercial products. In general, these products comprise textiles, filter, sealing and packing materials, asbestos boards and papers, asbestos cement, thermal insulants, brake and clutch linings, and asbestos plastics. Historically, it is probable that its first use was for incombustible lamp wicks and woven cloths.

CANADA

Canada was for many years the principal world source of asbestos, and still produces from 38% to 40% of the world total. The approximate production figures for all types of commercial asbestos during 1965 were:

Russia	1,500,000 short tons
Canada	1,400,000 short tons
South Africa	220,000 short tons
Rhodesia	150,000 short tons
U.S.A.	130,000 short tons
Sub-Total	3,400,000 short tons
Miscellaneous	100,000 short tons
TOTAL	3,500,000 short tons

The bulk of asbestos rock is produced from open pits, often of great size, but some mines, for a variety of reasons, operate underground workings.

Our client for this project, Bell Asbestos Mines Ltd., started as a predecessor company with an open pit about 1876 and converted to underground operations in 1951.

The asbestos at Thetford Mines, Quebec, Canada, where our client's workings are situated, is chrysotile and it occurs in a random pattern of closely spaced veins in serpentinized peridotite. The lengths of the individual fibres in the deposit vary. They can be minute or more than an inch long.

MINING

The caving method of underground mining is used by Bell Asbestos Mines Ltd., and by its neighbour, the King-Beaver Mine of the Asbestos Corporation Ltd. The same method was used for some years following World War II by the Canadian Johns-Manville Company Ltd. at Asbestos, Quebec, but this mine was reconverted to open pit operations in recent years.

Caving consists essentially of undercutting a column of ore and allowing the column to break down due to its own weight. The ore is taken away through 'drawpoints' at the bottom of the caving column, hauled in trains to a vertical shaft and hoisted to the surface for processing. The height of the column depends upon circumstances but is usually in the order of 300 ft. to 400 ft.

MILLING

The surface processing of the ore, or milling as it is commonly called, consists of drying, followed by several stages of

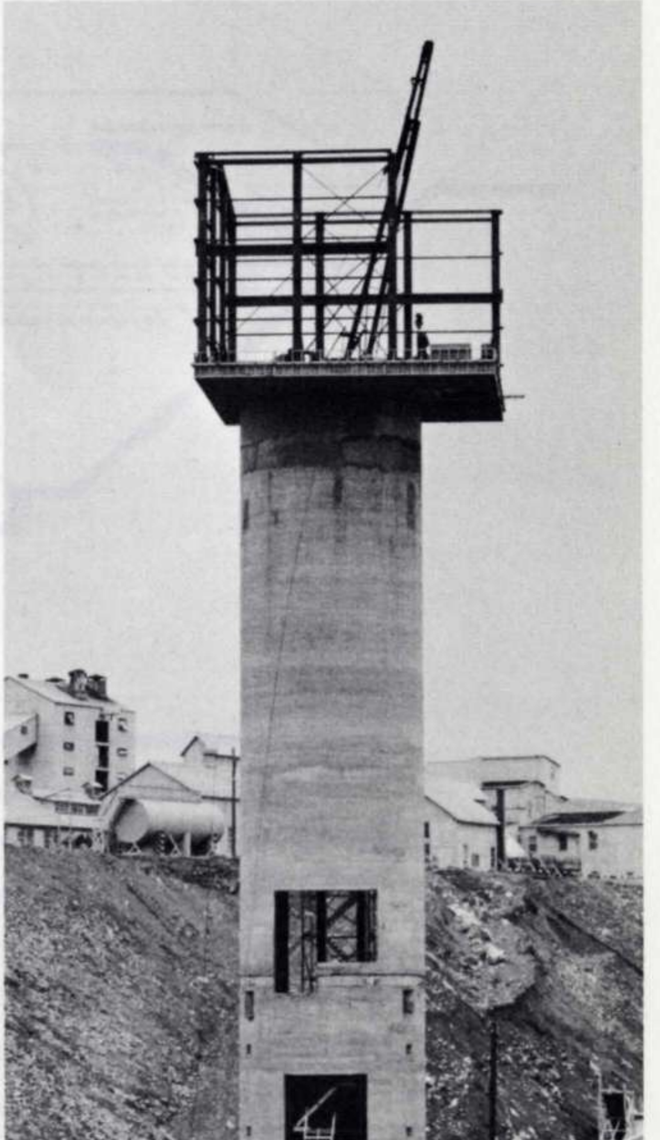


Fig. 1
View of tower during construction of penthouse before backfilling to underside of upper opening.
(Photo: Leigh W. Bladon)

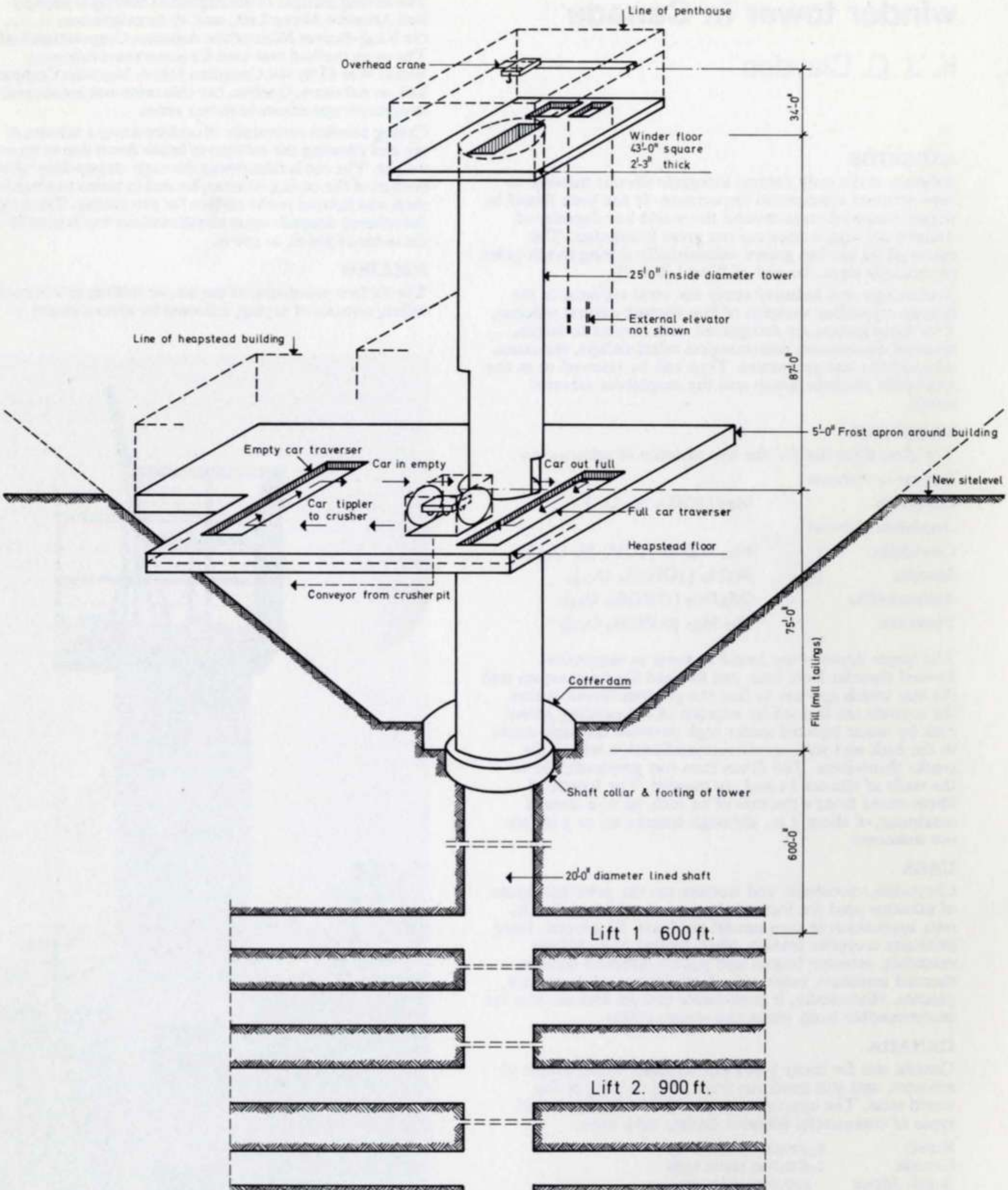


Fig. 2
Bell Asbestos Mines Ltd.
Project 50
Winder tower and heapstead floor

crushing to free the fibre from the host rock, each stage being followed by a combined operation of screening and aspiration to separate the freed fibre from the rock particles. The fibre is then classified into a number of close-tolerance screen analysis grades before being packed in bags for shipment.

The mining of asbestos ore by any method brings about some damage to the fibre, principally the opening and fluffing of the crystalline bundles and the absorption of rock dust by the opened material. This condition makes difficult the milling process and the classification of the fibre into acceptable commercial grades. Underground mining is rather more severe in this regard than is open pit mining, and largely for this reason, Bell Asbestos Mines Ltd. is installing a method of bringing the ore to the surface in mine cars directly from the working areas. The method is similar to the cage winding practice in British coal mines.

THE WINDER TOWER

Head Wrightson Stockton Ltd. was appointed to design, fabricate and commission the car handling circuits on the surface and on the underground levels and we were appointed to design the winder tower for a new shaft and the associated heapstead buildings and traverser floor.

The site of the new shaft is in an area previously used for the disposal of waste rock from the old open pit and of tailings from the mill. A substantial quantity of this material had to be moved to allow the winder tower to be collared in bedrock.

Initial design showed that the tower and collar could form one structure giving a cylindrical shaft about 150 ft. high with 1 ft. 3 in. wall thickness which would be very suitable for using sliding form construction.
The arrangement is shown in Fig. 2.

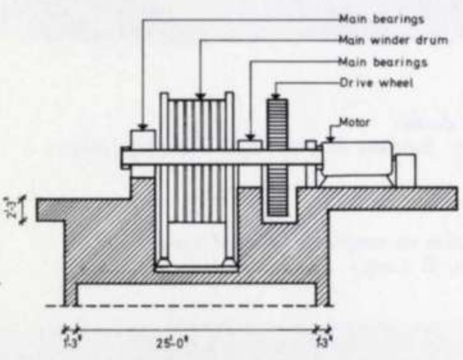
THE WINDER FLOOR

The winder floor, 43 ft. square, on which the main winder drum and motors will be mounted, is placed eccentrically on the top of the 25 ft. diameter shaft and is enclosed by a steel-framed building 34 ft. high. The machinery loads presented the major problem in design, the most critical conditions during mine working occurring either due to rope breaking on overwind or due to obstruction jamming the cage within the shaft. This condition results in two heavy loads of 2,400 kips applied by the bearings on either side of the main winder drum. Originally it was intended to use a traditional form of two main winder beams but during discussions with the client, he expressed a preference for a winder drum shaft at hand height above the floor, which depressed the beams into the slab, and we found the simplest solution was to use a 9 ft. thick reinforced plug in the top of the tower (see Fig. 3).
The plug was analysed as a two-dimensional plate—the stress in the third dimension being allowed for when interpreting the result. The cantilever of the winder platform was designed as a plate. The design of the mechanical equipment required that deflection should be kept down to small limits.

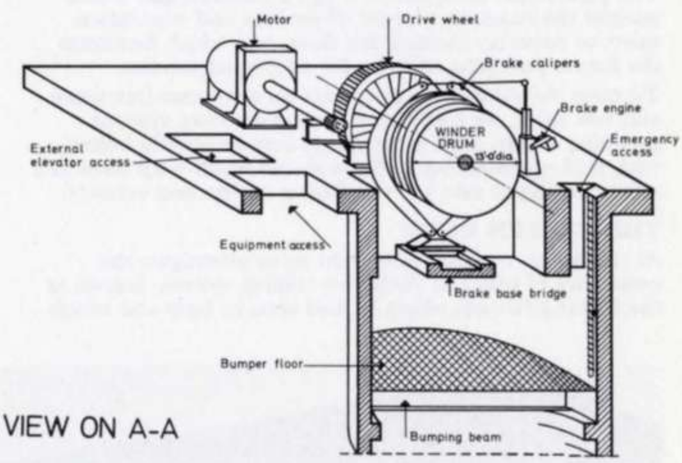
CONSTRUCTION

Construction started in March 1966 with the removal of surface dump and the installation of a coffer damp through the remaining 20 ft. of clay overburden.
The first 20 ft. of the shaft was driven prior to placing the tower foundation to ensure that subsequent shaft sinking should not damage the tower support. The shaft itself was constructed using sliding formwork, the operation taking 13 days. The programme allows for completing the tower structure before bad weather prevents further concreting. The winder machinery will be installed during the summer of 1967, to be followed by the sinking of the shaft.

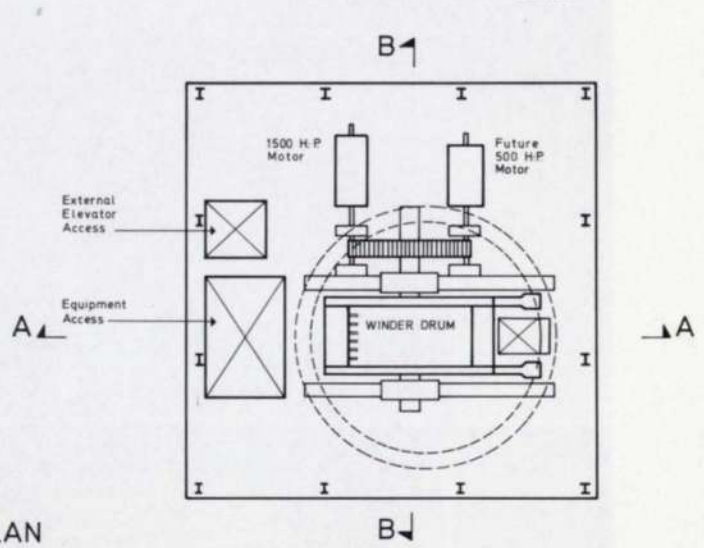
The tower was built by Barnett McQueen Co. Ltd., Fort William, Ontario, using sliding formwork lifted by air operated jacks. These they find more reliable than the more usual oil jacks. Construction of the platform followed the slide and a number of knee brackets fabricated from rolled steel sections were used to provide temporary support.



SECT. B-B



VIEW ON A-A



PLAN
Winder room floor

Fig. 3
Bell Asbestos Mines Ltd.
Winder tower—Project 50
Hoisting equipment in penthouse

Factory for Vitality Bulbs Ltd at Bury St. Edmunds

Joe Levy

CLIENT AND BRIEF

The name Vitality Bulbs conjures up colourful vistas of Dutch tulip fields—but not for long. These here bulbs are miniature electric ones such as you find in cars and radiograms. The client acquired a large site at Bury St. Edmunds and decided to move his business from some old and very overcrowded premises in Wood Green to pleasanter pastures. He engaged T. F. Morris & Partners of Cambridge who are experienced industrial architects and asked them to design him a factory which could be extended eventually into nine sections on a 3 x 3 pattern.

We were consulted at a very early stage and in fact visited the existing factory to see the manufacture of bulbs in progress. This is done by automatic machines which use large numbers of gas flames and consequently a large amount of heat is liberated. This poses a ventilation problem which is complicated by the fact that the production floor must be completely draught-free. The client also asked for the production area, which measures 102 ft. x 86 ft. to be column-free internally and on the two long sides (for future expansion).

GENERAL DESIGN OF STRUCTURE

The final design is a two-storey building, the production floor being above the ground floor offices and plant rooms. The production floor slab is a 15 in. coffered slab which enables the required amount of services and ventilation inlets to come up through the floor, and which facilitates the future punching of holes for additional services.

To meet the client's requirements on a column-free space and two sides, we considered several one-way systems spanning 102 ft. such as lattice-girders or castella beams, with roofing consisting of felt-covered wood-wool slabs and a false ceiling to take the ventilation outlets and extracts.

THE BEHLEN ROOF

At this stage, the architect asked us to investigate the possibility of using an American roofing system, known as the BEHLEN roof, which he had seen in Italy and which was made there under licence by a firm called Sicit. This consists of deeply corrugated galvanized mild steel sheets, varying in thickness from 22g. to 14g., used top and bottom and kept apart by lattice girders at 3 ft. 4 in. centres. The lattice girders are made up of cold-formed galvanized light gauge sections, and are stud-bolted at 3 in. centres to the corrugated sheets. The corrugated sheets act as stressed-skin members taking the tension and compression forces, while the lattice girders take the shear forces and provide the necessary bracing. By this means, clear spans of 180 ft. are possible using a 9 ft. depth. Also the ceiling is already provided by the structure and the roof space becomes valuable plant space.

Clearly such a roof would suit our purposes if it were economic. A preliminary check on prices showed that it was in fact cheaper than the traditional alternatives previously considered. The client then suggested that a small party of us could go Italy, visit a factory using the BEHLEN roof and discuss technical matters with the manufacturers. This was obviously the quickest way to come to a decision, so a group consisting of the client, his production managers, the architect, the quantity surveyor and myself went over to Milan for a couple of days.

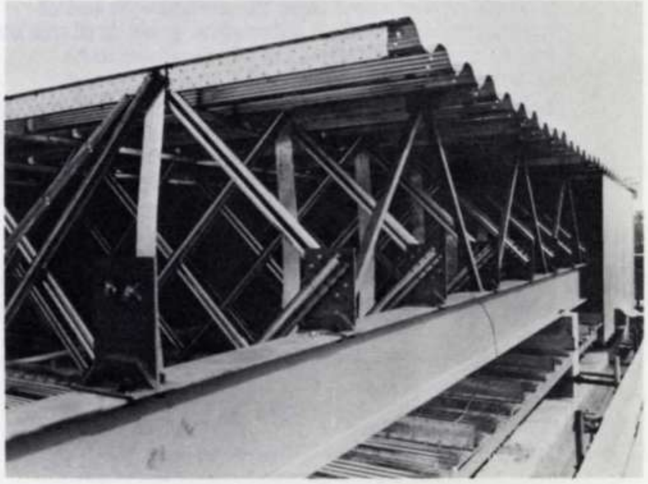
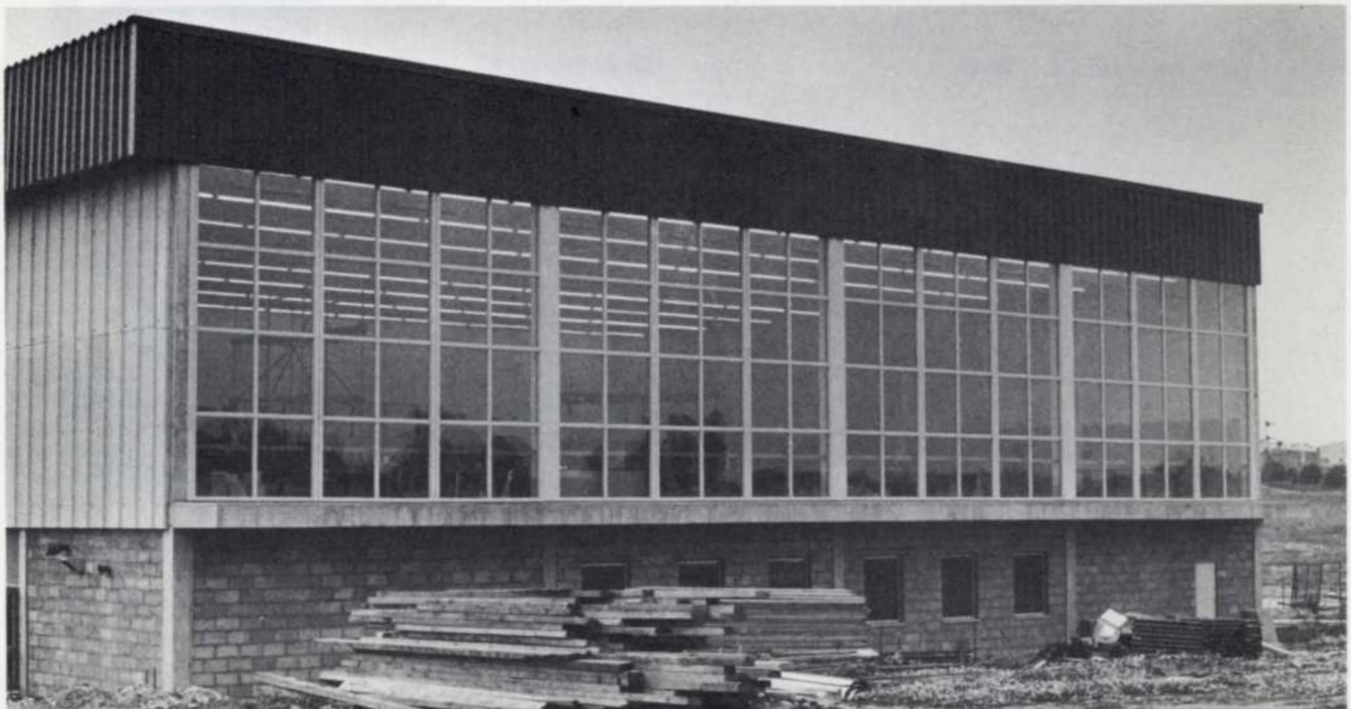


Fig. 1
Eaves detail.
(Photo: Boulton & Paul Ltd. Publicity Dept.)

Fig. 2
Elevation on temporary glazed end.
(Photo: J. Levy)



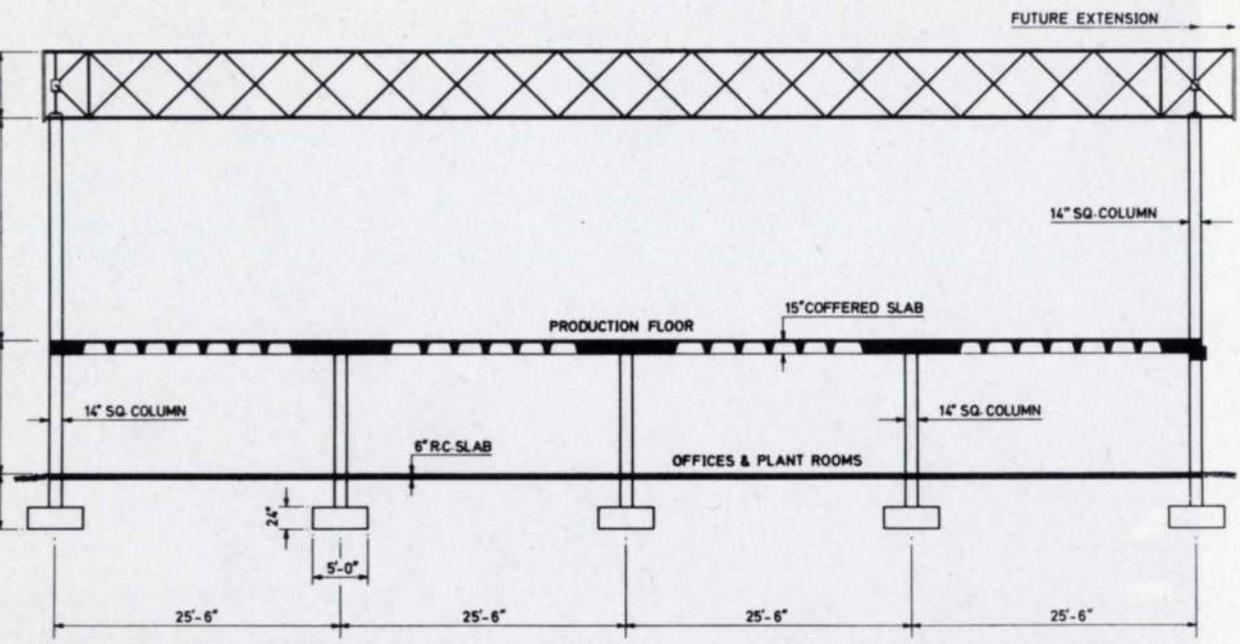


Fig. 3
Cross-section.

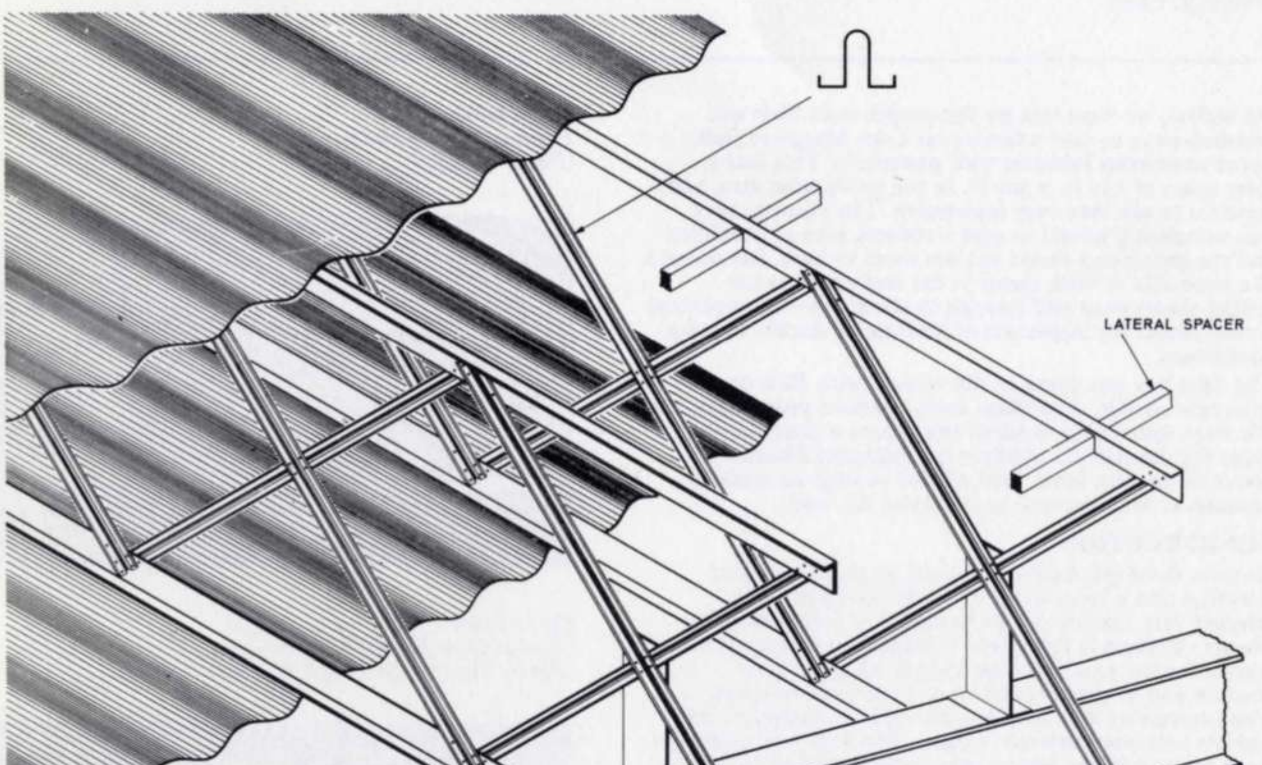


Fig. 4
General detail and support section.

Fig. 5
Ceiling view showing soffit steelwork.
(Photo: J. Levy)

On arrival, we were met by the people from Sicit and whisked away to visit a factory on Lake Maggiore (talk about combining business with pleasure!). This had a clear space of 150 ft. x 300 ft. in the production area, and needless to say, was very impressive. The climate there was sufficiently humid to give a realistic idea of durability and the galvanized sheets did not seem to have deteriorated. We were able to walk about in the roof space as the ceiling sheets were stiff enough to allow this—an important consideration for inspection of services or electric lighting equipment.

The next day was spent in discussions with Sicit on structural details, durability, costs, erection procedures, etc. We were sufficiently satisfied and placed a provisional order for the roof for delivery the following March—about six months later. Sicit offered to send an erection foreman at their expense to supervise the work.

CONSTRUCTION

As soon as we got back, we started preparing working drawings and a contractor, T. C. Stewart Ltd., was selected very quickly on the basis of a schedule of rates. Messrs. Boulton & Paul were interested in taking up the agency for the roof and were appointed as the roof erectors and as suppliers of other structural steelwork. The contractors took over the site early in January 1966, and the roof parts arrived in May. The roof was erected in three weeks and the factory was handed over at the end of August.

The design stresses in the roof members were in accordance with BS 449, but the thickness of galvanizing was only about half that recommended in BS 2569. We therefore recommended that all external surfaces should be painted with two coats of chlorinated-rubber paint for additional protection. The soffit was painted white and in conjunction with the strip lighting, gives a very even light over the production floor.

The temporary elevations of the building have removable structural steel members and are either fully glazed on the north side, or clad with asbestos cement sheets.

Working conditions are very pleasant and the client had no difficulty in attracting local labour. The ventilation system works very well and the hot air collected in the roof ducts can be re-circulated as part of the heating system.

Fig. 6 below
Erection of cross girder.
(Photo: J. Levy)

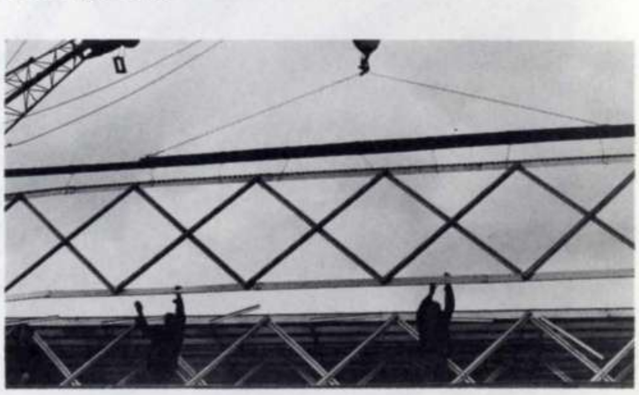


Fig. 7 below
General view of partly erected roof.
(Photo: Boulton & Paul Ltd. Publicity Dept.)

A lesson from the past—
The Tay Bridge Disaster

J. Shipway

As a change from the rain of current periodicals which constantly descend on my defenceless head, I recently spent an evening in the public library reading the '*Report of the Court of Inquiry, Tay Bridge Disaster, 1880*'.
The history of engineering has always rather attracted me and there is much we can learn from the past. Although everyone has heard of the Tay Bridge disaster, few know the facts of what perhaps can be described as the most tragic structural failure in British engineering. There is plenty that is relevant to current structural affairs in the events which led up to the fall of the bridge on the night of 28 December, 1879.

THE DESIGNER

The bridge was designed by Thomas Bouch (1822-1880), an experienced engineer with many successful bridges to his credit, including two in the north of England which were carrying railway traffic as recently as 1962. For most of his engineering life Bouch held the view that contemporary structures were overdesigned, and he introduced considerable refinement of material into both his masonry and iron bridges. It should be remembered that in the heyday of railway construction there were no codes of practice to offer permissible stresses, and structural design was often dominated by rule-of-thumb and practical experience rather than accurate calculation. Design in those days reflected the attitude and outlook of a particular engineer perhaps even more than it does now. Brunel was noted for well-proportioned bold designs, Stephenson's were more conservative. Like Bouch, they too had their failures, though not on such a scale.

The Tay is a shallow estuary, nearly two miles across at Dundee, with many sandbanks and a depth of water rarely exceeding 50 ft. Bouch planned a bridge to carry a single line of railway. There were to be 85 spans, 13 of which were navigation spans, higher and larger than the others. Eleven of these spans were 245 ft. and two were 227 ft. The remainder of the bridge had spans of between 67 ft. and 164 ft. The bridge deck was laid on top of the smaller spans, but ran through the girders of the navigation spans; the same arrangement holds in the rebuilt bridge today. In the disaster only the navigation spans fell. The remainder of the bridge remained standing but was dismantled later.

Borings in the river showed sand and gravel overlying rock at a depth of about 20 ft. below the river bed. It was planned to found each pier on the rock by caisson construction, and the piers themselves were to be of brickwork for their full height to the underside of the girders, which were a maximum of 92 ft. above high-water level. The girders themselves were in wrought iron, of the well-tested double triangular design so favoured by the Victorians.

THE CONTRACTOR

The contract was let to Charles de Bergue & Co. on 1 May, 1871. Work began from the Fife shore, and pushed out slowly into the river. After fourteen piers had been constructed it was found that the rock was no longer solid, and instead a shelf of conglomerate about 3 ft. or 4 ft. thick covered layers of coarse sand and fine micaceous sand, not always in that order. The borings had mistaken the shelf of conglomerate for bedrock, which in fact existed at increasingly greater depth as the distance outwards into the river increased. Caisson sinking proved extremely difficult owing to the presence of the conglomerate and random boulders. It was obvious that founding on rock farther out into the river would be impracticable and that the piers would have to be built on the sand.

THE PIERS

Bouch was apprehensive regarding settlement of the heavy brick piers, which he seems to have designed to apply about 8½ tons/sq.ft. on the rock. He apparently made no load test on the sand, but instead rapidly re-designed the piers to consist of groups of cast iron columns on a larger base, giving a bearing pressure of about 3¾ tons/sq.ft. on the sand. The cast iron columns formed a group of 6 in a hexagonal shape of about 24 ft. x 14 ft. in plan, the smaller dimension being in the line of the bridge. The batter on the ironwork was negligible and the piers gave the appearence of a spidery slenderness in their height of 90 ft. The columns themselves were 15 in. and 18 in. diameter, 1¼ in. thick. They were cast in lengths of 10 ft. and joined by bolting through flanges. Bracing was in the form of 4½ in. x ½ in. flats bolted at one end to lugs cast on the column. The other end had a jib and cotter connection to the lugs, to allow tightening.

THE NEW CONTRACTOR

About this time Charles de Bergue fell ill, and his firm was unable to continue the work and asked to be relieved of the contract. A new contract was let to a Middlesbrough firm, Hopkins, Gilkes & Co., on 26 June, 1874. This firm had previously done work for Bouch elsewhere and he trusted them. Accordingly, no specification was drawn up for the new cast iron work. This was to prove a fatal mistake. The wrought iron work in the girder spans formed part of the original contract and was covered by a specification, as was the brickwork of the pier construction.

INSPECTION

A temporary foundry was set up on the Fife shore, and casting got under way. Pier construction proceeded slowly across the river, the girder spans were floated out and erected, and by September 1877 the first train passed over. After work on the approaches was completed the bridge was inspected and passed by the Board of Trade. The Inspector was Major-General C. S. Hutchinson, R.E. The bridge was tested according to his instructions by running six coupled goods engines over it and measuring the deflections of the girder spans. The loading applied by the engines was nearly 1½ tons/ft.run. In his report Hutchinson observed, among other points, that

'*The lateral oscillation was very slight and the structure showed great stiffness.*'
'*The iron work has been well put together, both in columns and girders.*'
'*I would suggest 25 m.p.h. as a limit which should not be exceeded.*'
'*I should wish if possible to have an opportunity of observing the effects of a high wind, when a train of carriages is running over the bridge.*'

He made some minor criticisms regarding slack places in the rails and the preservation of the gauge, but that was all. The bridge was formally opened to traffic on 1 June, 1878. Queen Victoria crossed the bridge in June 1879 and knighted Bouch for his work. Sir Thomas was meantime busy with his design for a bridge over the Forth, and indeed foundation construction had begun on this site.

TROUBLES LOOM

All was not well with the Tay Bridge, however. The cast iron columns had been filled with concrete and were now cracking in various places. Contemporary accounts state the concrete was expanding and bursting the columns. The winter of 1878-79 was exceedingly cold and it was also said that the iron was cracking by shrinking onto the concrete.
Further, many of the lugs holding the bracing were snapping off, due it was said, to having been burned on afterwards instead of cast integral with the column. The bracing itself was loose in places owing to the cotter connection being vulnerable to vibration. Local enthusiasts timed the trains as they raced the ferry, and found they were travelling at over 40 m.p.h.

Bouch had the cracked columns strapped with iron bands, and seemed not to worry, since, he claimed, Brunel had had the same trouble at Chepstow. Other repairs were carried out to the bracing by the contractor's foreman, who (it transpired later) had an uneasy conscience about the casting and did the repairs at his own expense without informing Bouch. The bridge maintenance continued in this haphazard fashion and the railway company seemed unaware of the situation.

THE DISASTER

December 28, 1879 was a Sunday, and there sprang up on that day one of those tremendous westerly gales which occasionally sweep the central plain of Scotland.
The 7.14 p.m. train from Edinburgh, consisting of an engine and six coaches with 75 passengers on board, passed punctually onto the bridge. Its lights were seen to disappear,

but for a time no-one realised what had happened. Later the station staff from Dundee made their way out onto the bridge in the darkness, and discovered the navigation spans had fallen into the river, taking the train with them. There were no survivors.* The disaster gripped the public's imagination as no other has done since.

The 13 navigation spans had been of continuous construction, arranged in groups of 5, 4 and 4 reading from the south. The engine and coaches were found inside the fourth and fifth spans of the first group, the throttle of the engine being fully open and suggesting that the driver had had no warning. The film '*Hatter's Castle*', if I remember aright, showed the bridge being blown down before the train came on it, but this was obviously artist's licence.

EXPLANATIONS

Various theories were put forward to explain the disaster. Inspection of the shattered piers showed the ironwork to be almost indescribably bad, with patched blowholes, uneven thickness of metal in the walls of the columns and boltholes for connecting flanges and bracing cast in by the use of tapered formers, instead of being bored true after casting. These conical holes had obviously given very poor bearing for the bolts, and it was no wonder that play in the fastenings had occurred. Fissures in the columns had been filled in and disguised by a mixture of iron fillings and lead, and painted over. The holding-down bolts connecting the columns to the piers were sufficient to take a load of 200 tons, but they were anchored in only two courses of masonry, which at most gave a resistance of 6 or 7 tons.

It was significant that the spans were found in the bed of the river only about 30 ft. away from the piers. Since the piers were 90 ft. high they might have been expected to throw the spans this distance as they fell. It was deduced that the piers has instead crumpled like a falling chimney owing to the fragility of the ironwork.

An interesting theory put forward by the former resident engineer was that the weight of the train in the penultimate span had caused the end of the continuous section to lift off its bearings sufficiently for the wind to blow it off the pier. Since the engine was found in the end span, it was assumed its momentum could have carried it forward to that position during the fall. The details of the bearing showed no holding-down attachment, and this theory looked possible. Surprisingly enough, it is supported by a current vice-president of the Institution of Civil Engineers, H. Shirley-Smith, in his book '*The World's Great Bridges*'. A simple calculation will show, however, that since the dead weight of the span was around 500 tons, the live load in the penultimate span necessary to reduce the end reaction to zero is about 4,000 tons, assuming full continuity under dead load, which was unlikely to have been achieved. Even the test load on the bridge totalled only 370 tons per span, and the train in the disaster must have weighed considerably less. Further, had the girder slipped off its bearings and brought down the pier in its fall it would obviously have been found in the river close to the base of the pier instead of some distance away. It seems unlikely, therefore, that this feature contributed to the disaster.

WIND FORCES

A court of inquiry was set up, the panel consisting of W. H. Barlow, then President of the I.C.E., Colonel W. Yolland, Inspecting Officer of Railways, and H. C. Rothery, Commissioner of Wrecks. The most significant fact to be exposed by the Inquiry was that Bouch had taken little notice of wind forces in his design of the structure. The Astronomer Royal had suggested a wind force of 10 lb./sq.ft. but Bouch must have known that the French were using a figure of 55 lb./sq.ft. at that time, and the Americans 50 lb./sq.ft. His theory was that the train had been blown off the rails, and had so damaged a girder that it had collapsed and brought down the piers with them. He supported this with evidence that the last two coaches were much more seriously damaged than the remainder of the train. This seemed an improbable cause, but the girder was raised from the river and inspected. It showed little more than score marks.

**There was a survivor. This was the railway engine. It was retrieved from the bed of the river and had a useful life for many years afterwards. It was known among railwaymen as 'The Diver'.*

Fig. 1
Engine inside the navigation spans of the old Tay Bridge. Tie members are unbraced flats with unusual rivet pattern of horizontal rows.

Various calculations were made to determine the wind pressure necessary to overturn the bridge, assuming all the ironwork was sound. It was calculated that without holding-down bolts, 37 lb./sq.ft. would have been required with no train on the bridge. With the train, 34 lb./sq.ft. would have been necessary. With the holding-down bolts as built, the pressure required increased to about 40 lb./sq.ft. Under this pressure the stress in the bracing flats would have been as high as 23 tons/sq.in. When tested after the accident specimens of the bracing failed in a range between 21 and 25.6 tons/sq.in. Even had the wind in that exposed estuary exerted only 20 lb./sq.ft., the factor of safety was obviously too low.

THE END

When the findings were issued Bouch was severely censured. It was stated the bridge was badly designed, badly constructed and badly maintained. Supervision had been insufficient at the foundry, and the bracing and fastenings of the columns were inadequate. The narrow base and lack of batter of the piers were also criticised.

Increased vibration had resulted from trains crossing the bridge at 40 m.p.h. instead of the recommended 25 m.p.h. and the railway company were blamed for this.

Sir Thomas retired to the little town of Moffat, Dumfriesshire and died six months later, a broken man. One has a certain sympathy for him, since his bridge was passed by the Board of Trade, and he had not been informed of the defects discovered in the bracing. The contractor's shoddy workmanship obviously played a large part in the failure. The design of his Forth bridge was abandoned and work ceased, but to this day the one completed pier can be seen off the island of Inchgarvie.

The iron girders of the remaining spans of the bridge were lifted off the old piers and re-used in the new bridge, rebuilt 60 ft. upstream from the old. As might be expected it is a heavy, undistinguished design, carrying a double track and twice the width of the old bridge. It was designed for a wind pressure of 56 lb./sq.ft., the bearing pressure on the sand being a maximum of $3\frac{1}{2}$ tons/sq.ft. The old piers of the bridge were taken down to 5 ft. above high-water level and left standing, where they can still be seen, a sombre and silent memorial to the tragedy of 1879.

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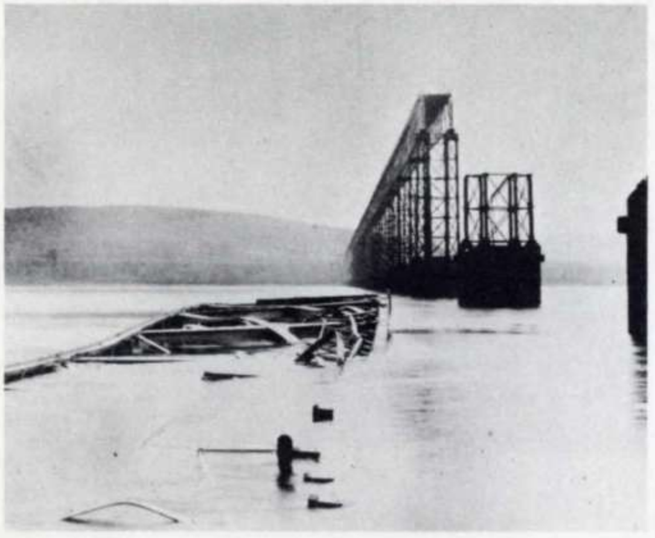


Fig. 2 above
After the disaster—view south towards Fife.

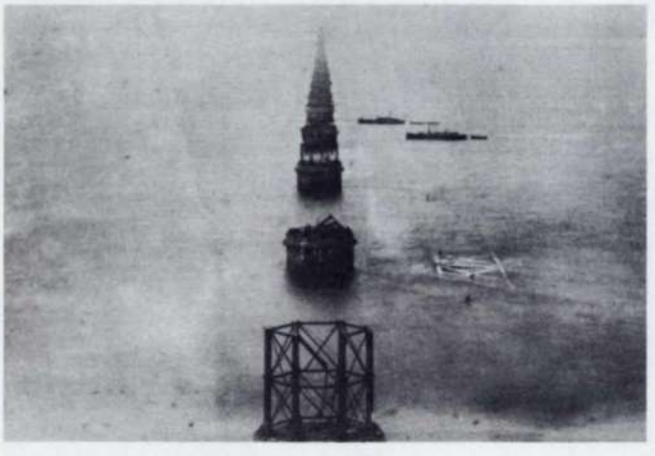
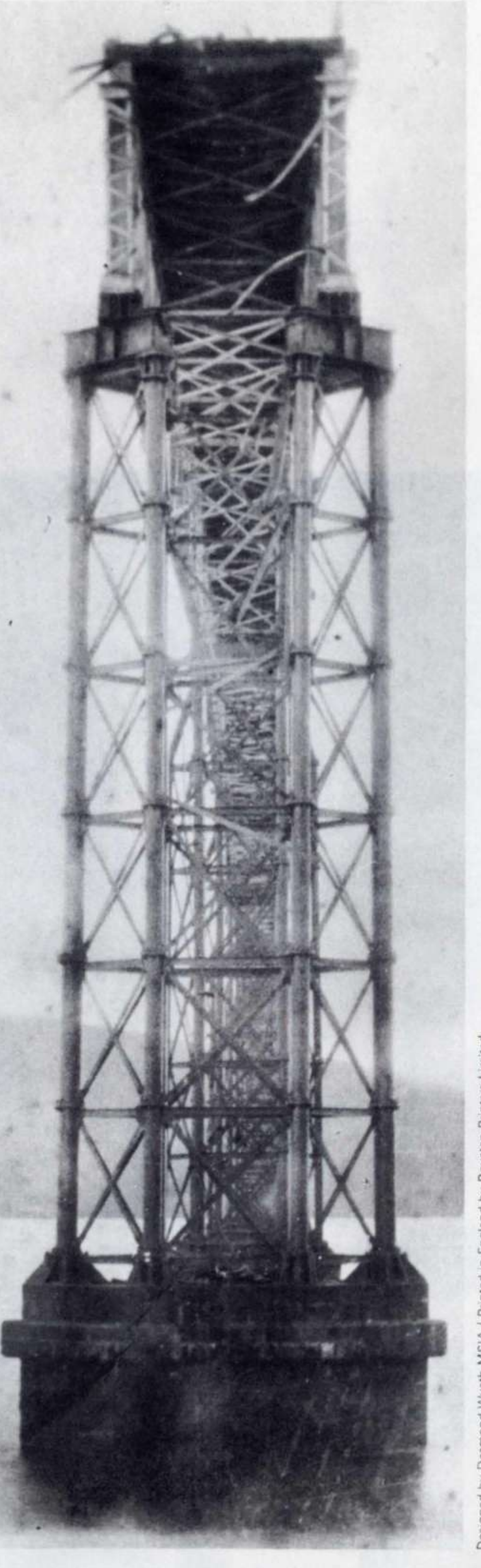


Fig. 3 above
The remains of the piers of the navigation spans. Nearly all the ironwork broke off at the masonry base.

Fig. 4 above
Lifting the remaining girders off the old brick piers for re-use in the new bridge. The new double-leg piers contrast with the narrow height of the old single-line bridge.

Fig. 5 right
Typical pier re-designed by Bouch in iron, 90 ft. high. The cast iron columns are in 10 ft. lengths, with ribbon-like flats for wind bracing.

All photographs for this article were taken by J. Shipway



Designed by Desmond Wyeth MSIA / Printed in England by Roystan Printers Limited.