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Contents

The front cover shows an elliptical stained glass window in shell roof of Sussex University Meeting House (Photo: E. Mark)
On the back is a general view of Sussex University Meeting House (Photo: Sussex Photo Agency)

Simple working drawings

Bryan Seymour*

A very large part of our work involves the preparation of working drawings. These are the drawings used for communicating information to many people who have varying functions and capacities. Although building methods and techniques have changed and are changing, the type of drawings being produced by consultants has changed very little.

SIMPLIFICATION
The current prefabricated concrete garages may well cause raised eyebrows over aesthetics but they illustrate very well why working drawings must be simplified. Consider the man who buys a prefabricated concrete garage for his own use. Normally he knows very little about building and even less about precast concrete. The main reason he buys it, apart from cost, is because the brochure showed how easy it was to build and that a matchstick man could start putting it up after lunch and finish in time for tea. The manufacturers supply all the necessary working drawings. To start with, simple instructions are given for the 'base' - there is no point in calling it ground floor slab and foundation. The manufacturers do not know the technical capacity of their clients so they must assume that it is very low. All working drawings, therefore, must require the minimum of technical experience. They must be very simple and show only what is required. The only instruction that is necessary is to show how the components go together, so all that is required is an assembly layout. The components are completely prefabricated so there is no need to give manufacturing details. The chances are that they would not be understood anyway.
The operative, who makes the concrete units and the client, require different information. The basic components are produced without any knowledge of their final assembly permutation. The man making the unit may be required, in simple terms, only to fit the reinforcement into the mould and then fill the mould with concrete. He wants to know only the details of the unit that he is making. Another reason why different types of drawings have to be produced is the fact that the final assembly layouts are drawn up after the components are made. The designers could have supplied one large comprehensive drawing to all those involved in the production and erection of each particular garage. Probably the client would not be able to understand this highly technical drawing and the design staff would have to produce a similar comprehensive drawing for every new garage instead of the simple line assembly diagram with standard photographically reproduced assembly details. The fabricator would have an almost impossible task comparing many complete schemes, trying to establish not only what different types of prefabricated units there were but how many there were. If there are, say, four types of garage using one particular unit and on each drawing, for each garage, these units are shown in detail, it is only after checking each of them with one another that it can be established that they are in fact all the same.
This simple example of building a garage, together with all the necessary techniques and organization, is a very modest form of system building. The same garage could well have been built in a more traditional way. For this construction a comprehensive drawing would normally be supplied but a skilled man would be used to build the garage and part of his skill is the reading of building drawings. The simple part-drawing method could equally be used for preparing the traditional building drawings. One of the main reasons why normal drawings are not always completely clarified is because detail can always be sorted out or corrected by the man on site. With prefabrication methods there can be no element of chance. The parts must fit when they are put together.

REAPPRAISAL
A completely unqualified and possibly non-technical man must be able to construct a complete, permanent garage in a day with the help of a neighbour and a very limited number of everyday tools. An answer is the prefabricated concrete garage, together with the necessary organization. We may not be concerned with producing simple precast garages but this rather feeble example has not only all the subtleties of our developing industry but possibly has an answer to some of our problems of manpower.
Our involvement in a more systematic form of building gave us the opportunity of reconsidering our method of preparing drawings and also the type of staff required to produce them. Although our drawing techniques evolved for a particular type of construction many of them are generally applicable. The building industry has for many years now used components, such as pre-made doors and window frames but the emphasis of system builders is to develop towards larger and more sophisticated components to such a degree that the complete building itself is becoming a component form. Our own dwelling unit is a component part of an overall complex, our house is part of a street, which is part of a district, which is part of a town or county, which is part of ... and so on.

DEVELOPMENT
Now consider just the paved area on the terrace of a typical house. It is to be constructed from standard precast slabs. A normal drawing (fig.3) would be drawn to show the required layout, sizes and details of all the slab units to be used, together with the necessary construction joint of slab to slab - in fact, a comprehensive drawing sheet

*All illustrations by the author who was helped with two of them by Jenny Birch

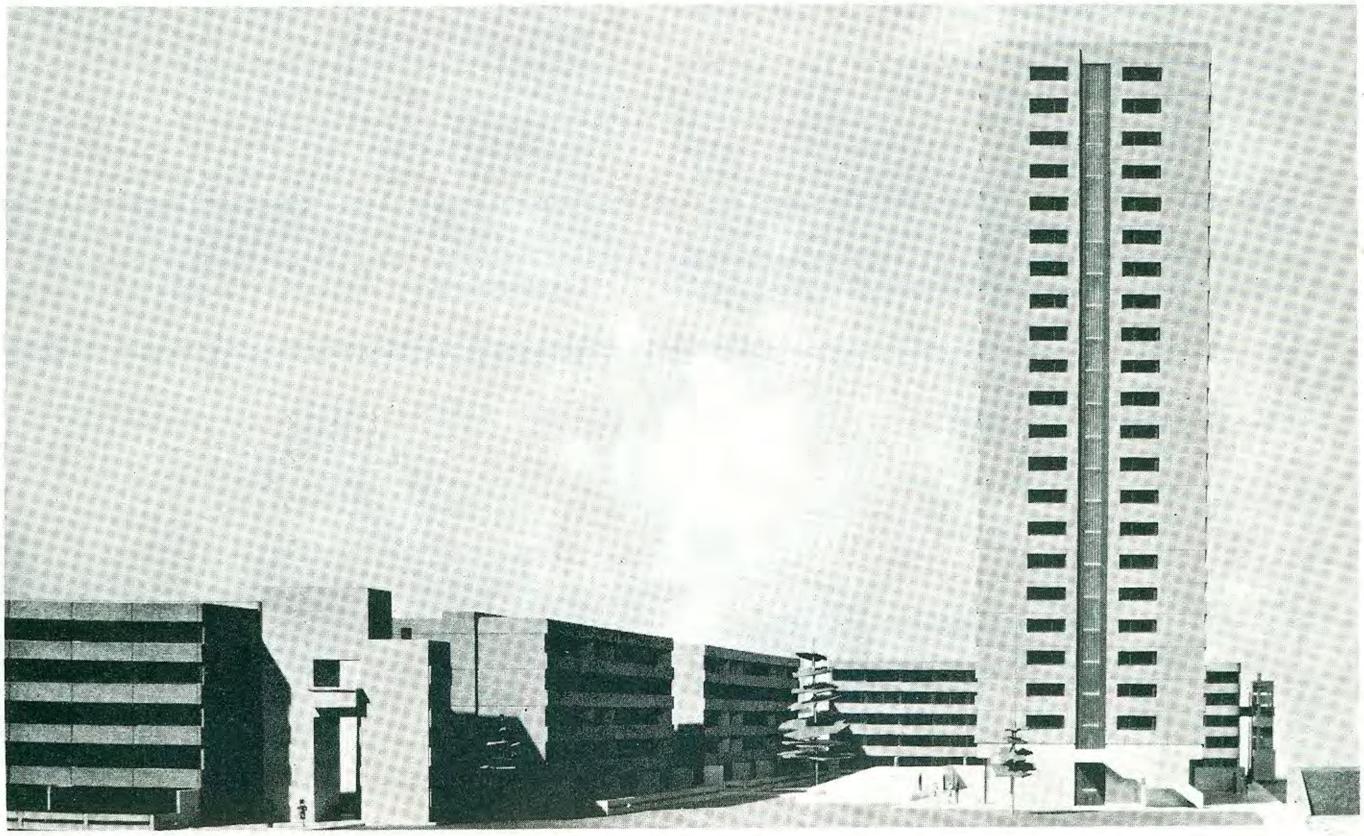


Fig. 1. above
King Edward's Road Mixed Development
in system building
(Photo: Henk Snoek)

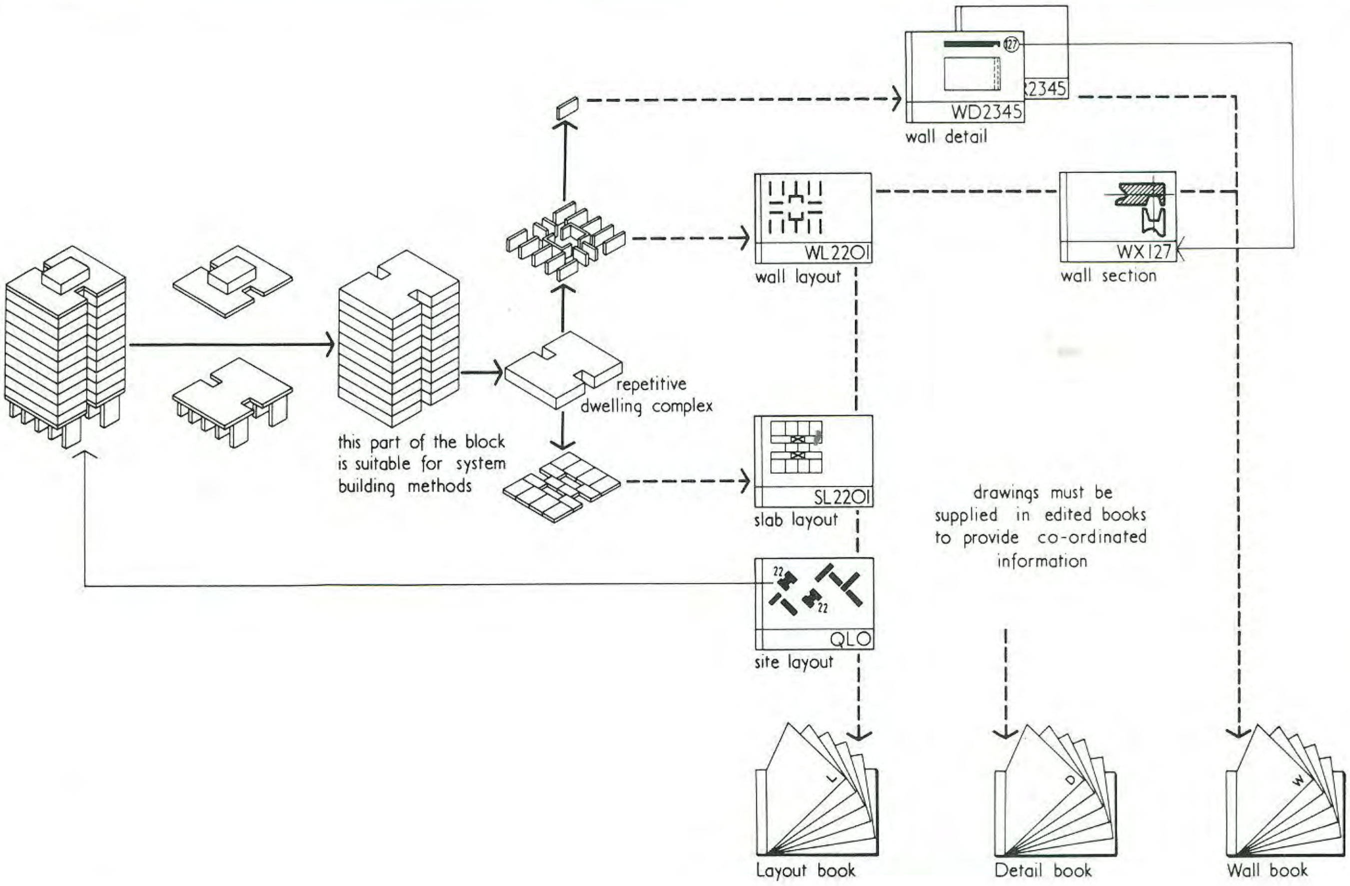
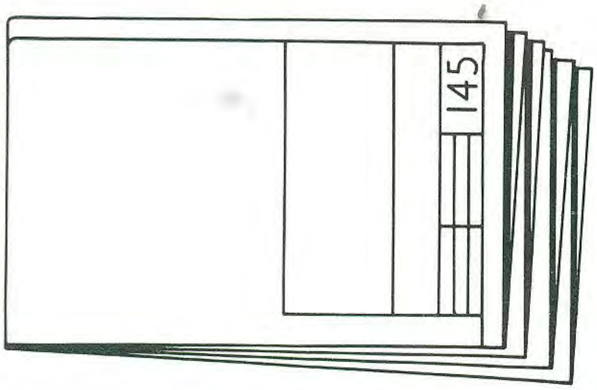


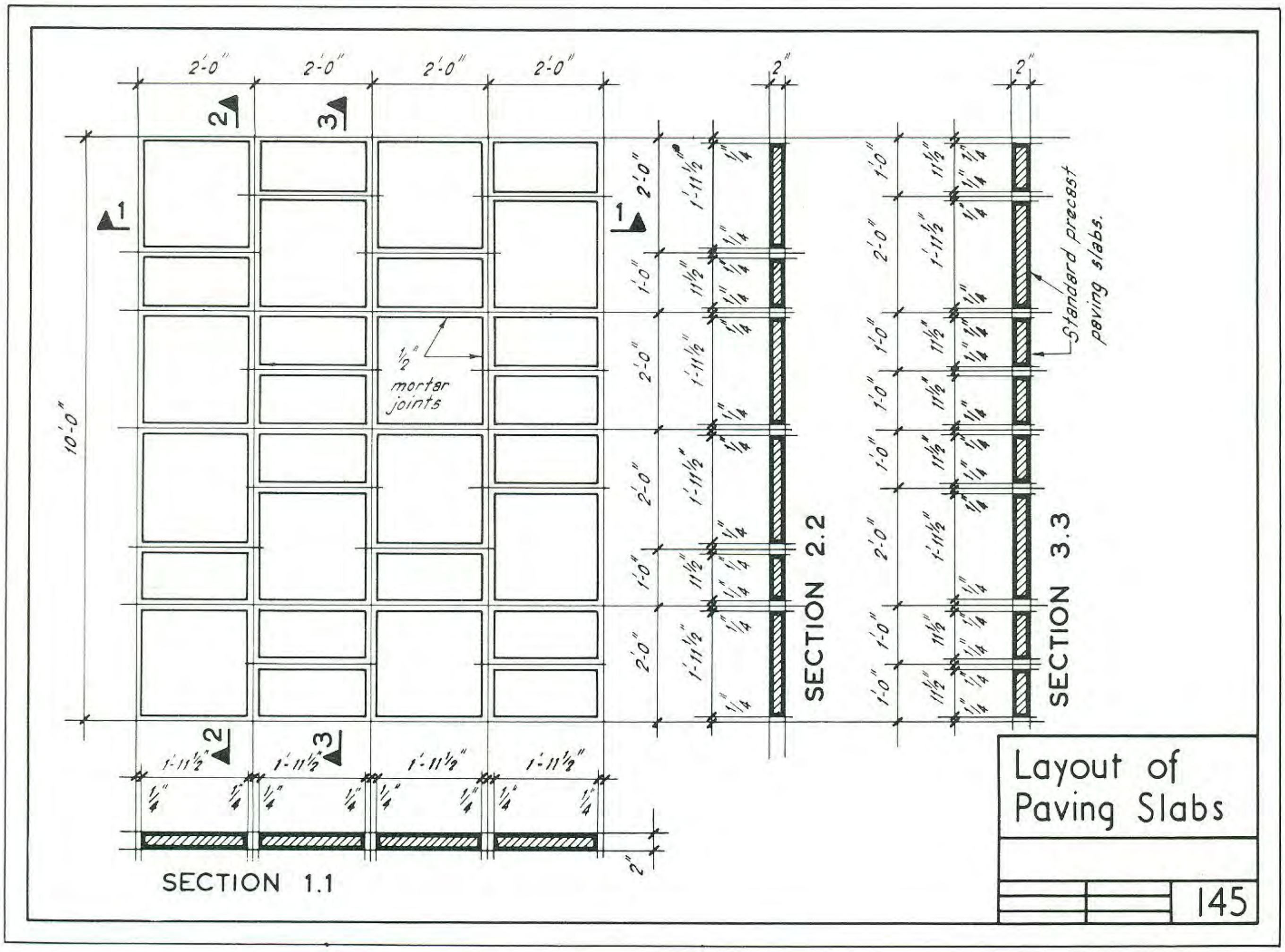
Fig. 2. Illustrates block of flats

showing everything. The details of units and construction will be taken from the manufacturer's working details that were used to make the standard paving slabs, therefore, the only new information shown on the large drawing is the layout of the slabs. When another terrace arrangement is required, the whole series of small drawings is repeated on another large drawing sheet. Another way to present the same information is, as they say in the army, 'by numbers'. The precast paving slabs are going to be designed long before this particular terrace is thought of, therefore separate drawing sheets are prepared for each different unit. Then the standard drawing sheets can be either extended or decreased in number as the range of units changes. Each unit will need a code number. This can also be given to the sheet showing that unit. For instance, the code number of one of the paving slabs and also its drawing code number is SD3. The only point to note about the code number at this stage is that the 'D' means Detail. Secondly, standard construction sections must also be drawn to show how the units go together. The standard sections can also be coded, for instance SX2. Here the 'X' indicates Section.

Fig.3.(continued on facing page)



Normal drawing



CO-ORDINATION

Breaking things up into component forms immediately presents the problem of showing how they fit together again. The white lines on a parking lot are the basic grid lines for co-ordinating the motor cars. Natural grid lines will always exist on layouts, column centres, wall positioning etc. All that is required is that they should be rationalized into a preferred pattern. The basic sizes of our paving slabs will indicate the grid reference that should be used in this case. In addition to the line co-ordination, a reference number is required. The slabs and standard sections already have code numbers so this can be extended to include the layouts, for example SL11. The 'L' indicates Layout. Once everything, that is to say, each component form, has its own code number, there is no need to draw it in detail again but only to show it diagrammatically on subsequent drawings with its coding. Not only is drawing then simple and quick but also the inevitable revisions can be made more easily. The drawings SL1001 could exist as a basic negative that was a standard drawing, showing only the component arrangement SL11. When a complete scheme is required using that basic part layout, a photographic negative could

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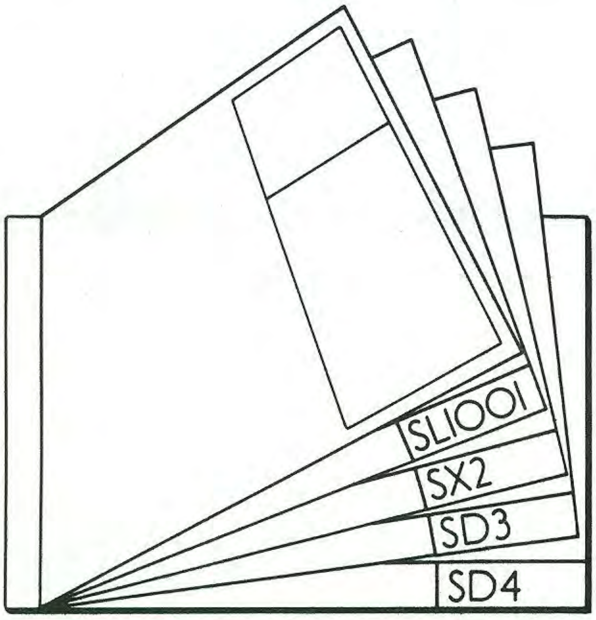
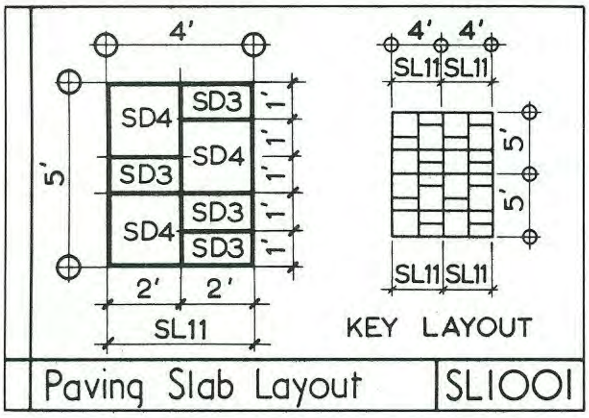
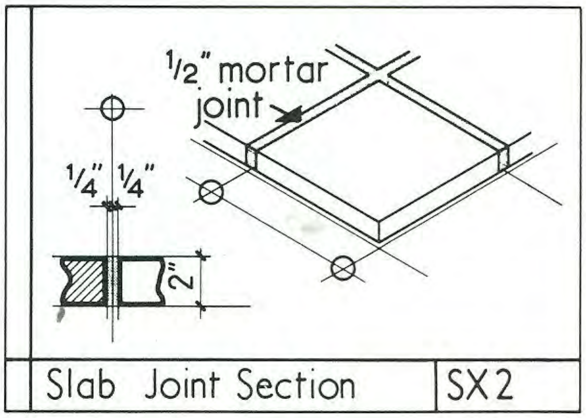
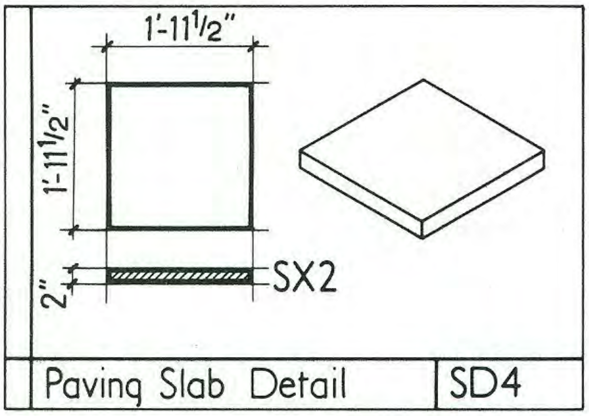
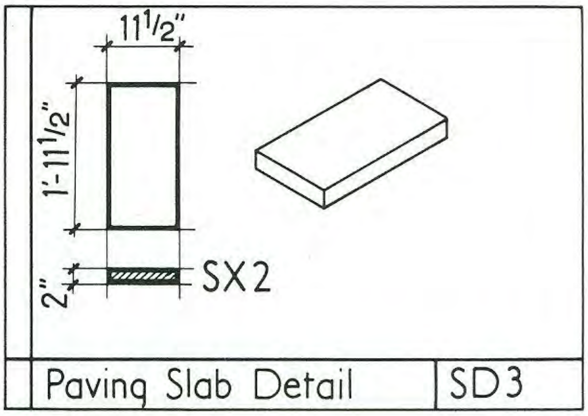


Element drawings in edited book









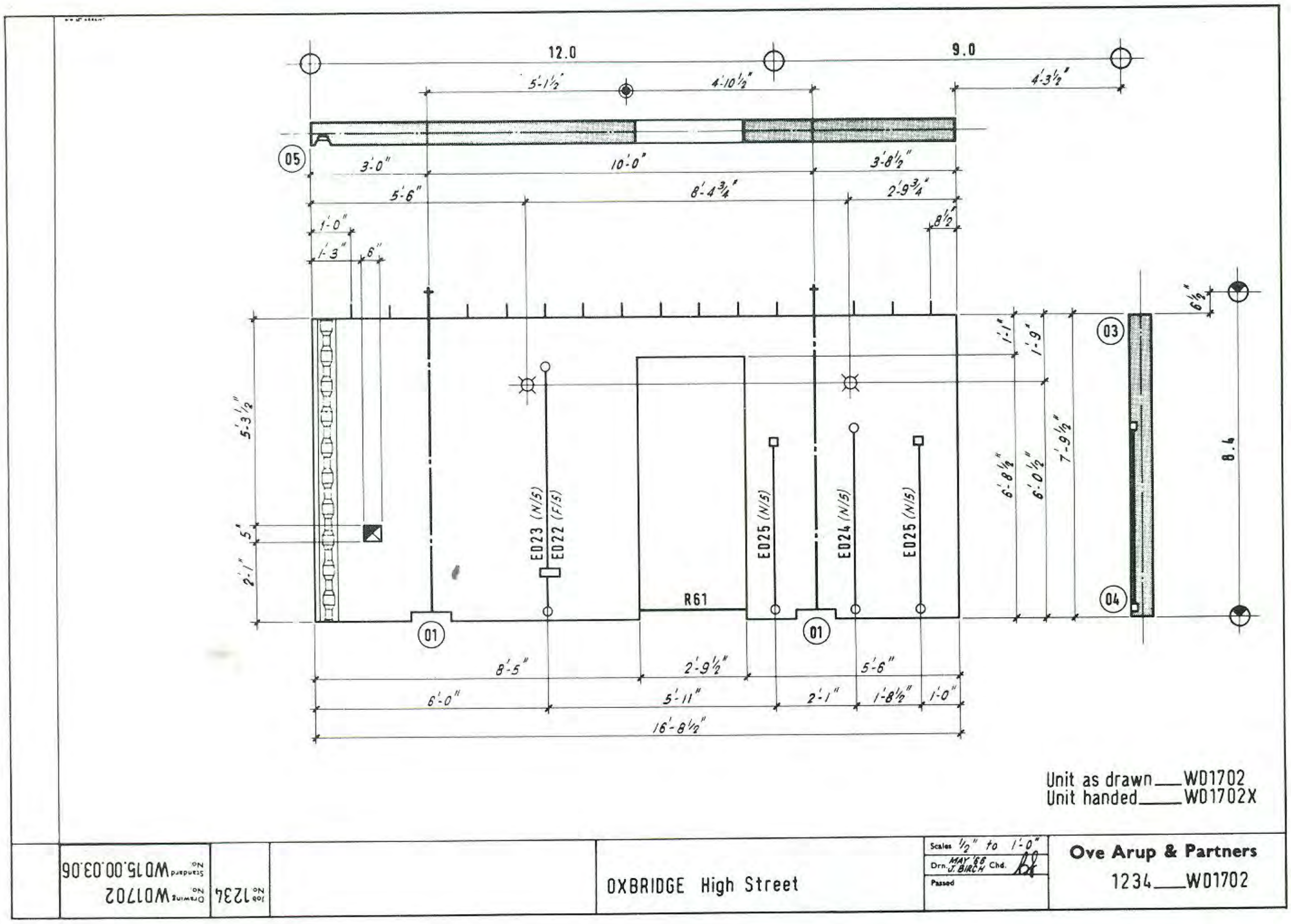
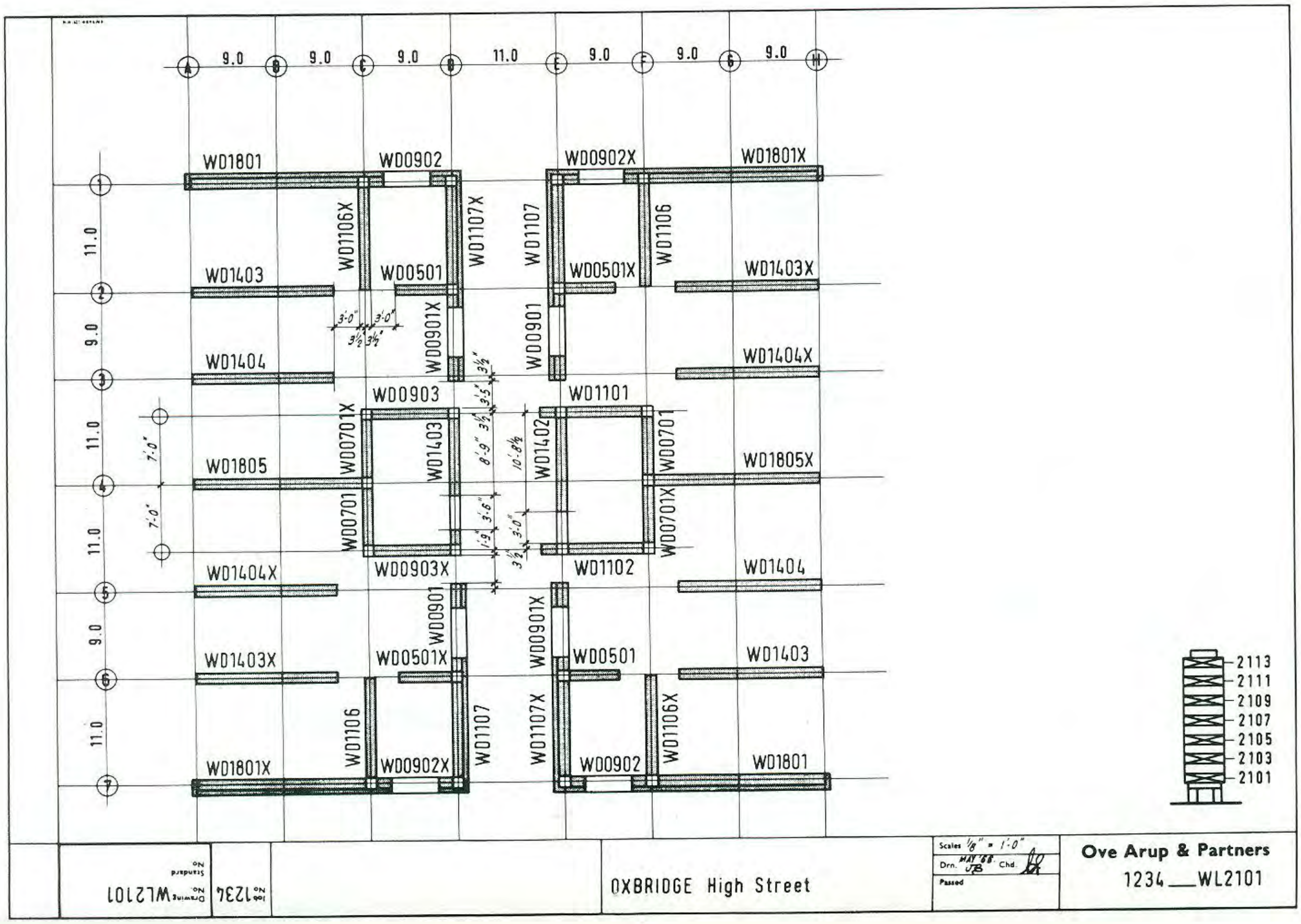


Fig.4. above
Typical component drawing

Fig.5. below
Typical layout drawing



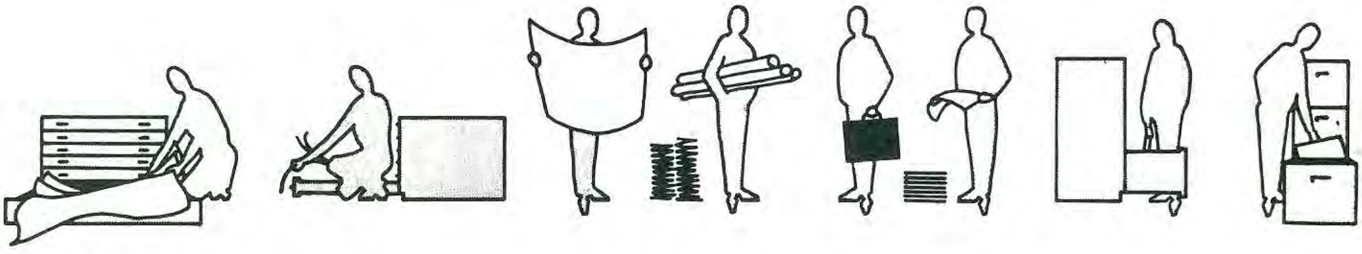


Fig.6 above
Advantages of small drawings

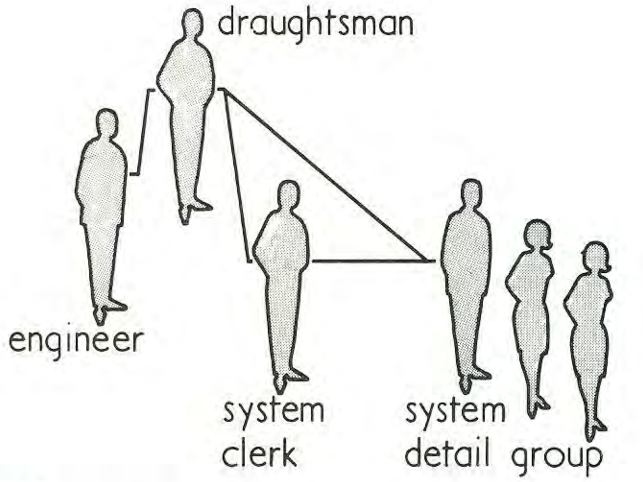


Fig.7
Pattern of group working

be produced and the key drawing SL1001 could then be added to the negative. Its number would then assume that of the key layout, i.e. SL1001.

SYSTEM BUILDING
The step from the paving slabs to the development towards industrialization is simple. Enlarge the precast paving slabs into dwelling-room size, make them thicker and add some 7/8 steel reinforcement and you have a form of system building. The illustrated block of flats (fig. 2) is this type of system. It has three basic parts. On the ground there are the main entrance and service areas, on the roof there are other service requirements such as lift-motor rooms and in the middle there is the repetitive living accommodation. This latter part is very suitable for system building methods and can be considered separately as a stack of repetitive layers. As each layer is identical, only one need be considered. The layer is a flat box with internal divisions but has no lid. The next box over will provide the lid to the box under. The box can be put into two basic groups of parts. Firstly, the sides and divisions are in fact the walls, cladding and partitions and secondly, the bottom of the box is the arrangement of floor slabs, stairs and half landings. When either of these groups of components is drawn it is given a code number and considered as component plans that repeat vertically. Other forms of plans could repeat horizontally. The thing to note is that the layouts show only the positioning of the prefabricated components and general assembly detail. If more specific detail is required then the code number will indicate the actual component drawing that must be referred to.
All drawings are cross-referenced automatically by the number coding technique. Information is compiled by the editing of a required set of drawings into book form. For example, the foreman erector will require a book of layout drawings whereas the factory manager requires a book of precast component details. Organizing methods gradually evolve within the drawing office to help the technical staff to do their jobs more efficiently.

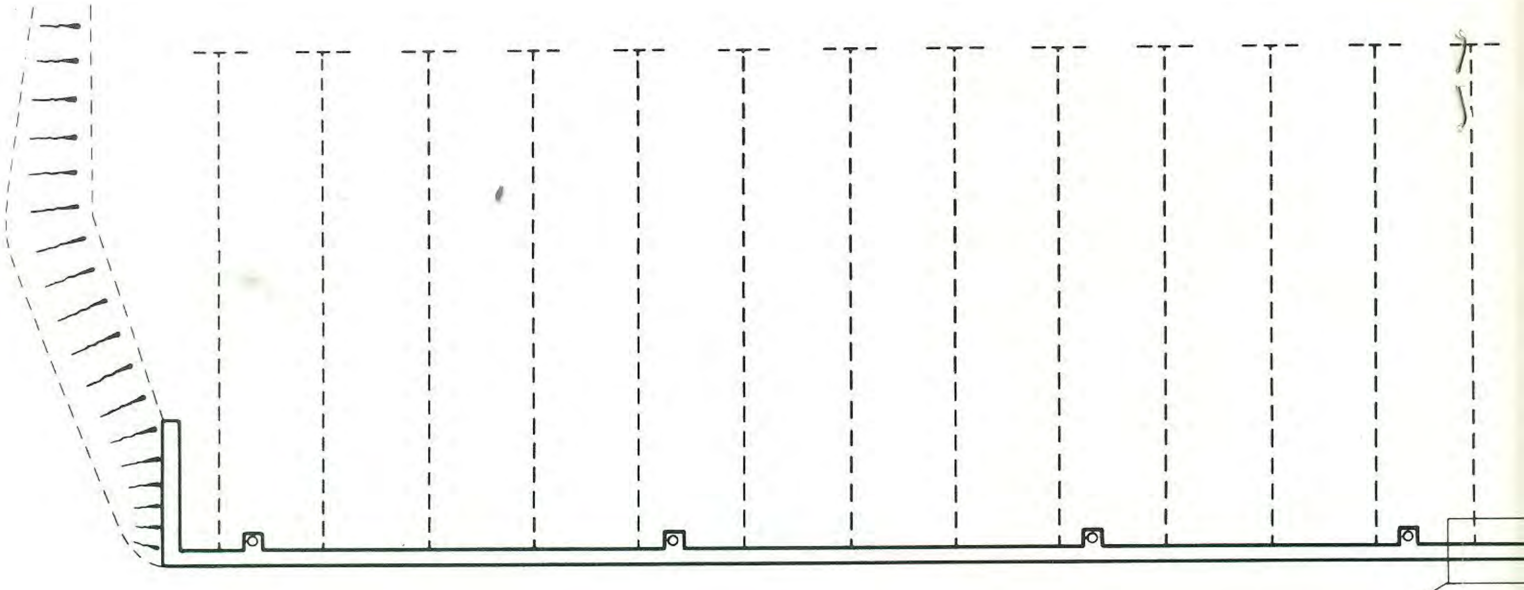
DRAWING OFFICE
Normal drawings require comprehensive skills not only to read them but also to produce them. This means, therefore, that a highly technical man may be using his full capacity for a very short time only. With this developing drawing technique, work can be split into groups which have a graded technical content. This allows the engineer to do engineering not draughting, which is particularly important with the younger engineers, who do not want to be draughtsmen. This kind of development is not particularly new. The structural steelwork industry has worked for many years in a similar way with designers, draughtsmen and detailers. The main problem is to translate the complex engineering requirements into a simple form so that they can then be easily read and drawn. A type of 'system drawing' shorthand is being perfected so that engineering staff do not spend as long explaining detail as they would in doing the details themselves.
In addition to the actual preparation of drawings there are many non-technical chores that can be taken from the technical staff to be done by non-technical clerks.
A pattern of group working is shown in figure 7. A job engineer has a draughtsman who processes the work into a 'draft' form. This will be a small job complete in itself, that is, self-explanatory, using the drawing shorthand methods. Having work that can be easily put into small job lots helps with the programming of work. The drafts are then passed to a 'system clerk' who books in, records programming requirements and performs other similar non-technical administrative duties.
A 'system clerk' does all the system processing. This consists of many activities, which include records and filing, organizing of printing, editing and despatching drawing books and programming.
Many general office aids are incorporated, such as vertical filing cabinets for the storage of drawings and coded file signals to support the numbering method.
Finally, the drafts are passed on to the System Detail Group so that the drawings can then be prepared. The pattern of group working (fig.7) is enlarged so that the System Administration and Detail Group will include other job engineers, engineers and draughtsmen, the whole then being co-ordinated by group control and development.

CONCLUSION
The actual work on site is only part of an overall process. All methods of systematization and industrialization on site must be fully supported by a similar development in planning. Although repetition and simplicity should be the common objective, complete co-ordination must be maintained by comprehensive planning. All component forms must not only fit together but it must be shown clearly how they fit together. Building, especially house building, must become economic, quick and efficient, or soon we may not be able to afford the required living standard and environment that are so desirable.

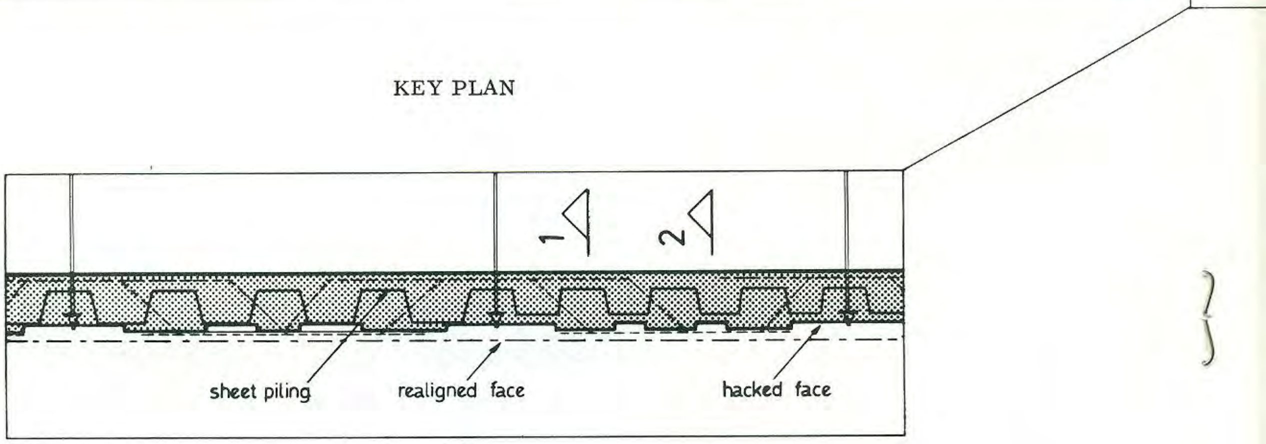
Defects and remedial work for new wharf at Littlehampton

B. P. Glover

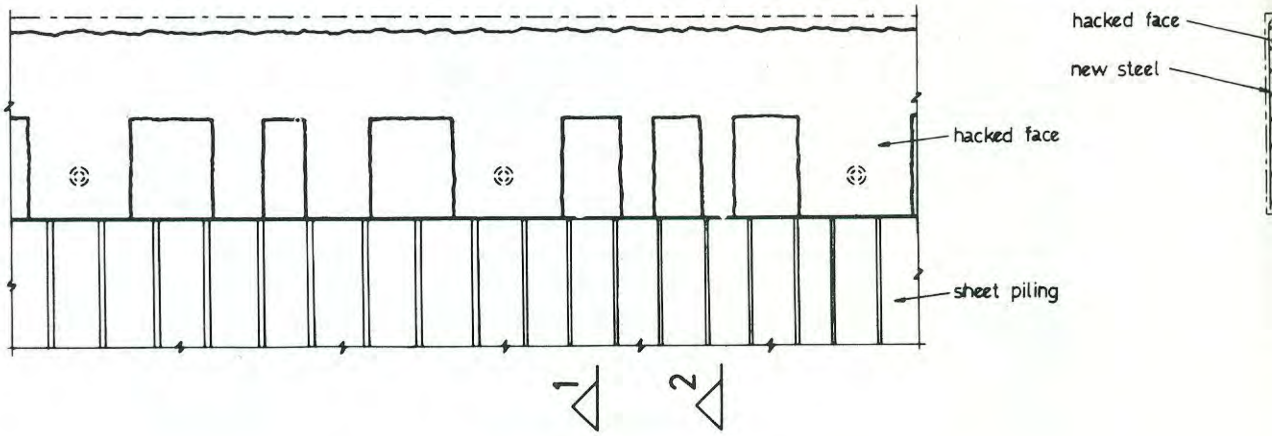
They say that experience is the best teacher, It should be, as very often it is the most expensive!
Most problems are satisfactorily solved if given sufficient forethought. However, there are occasions when things do go wrong. When this happens with a structure the remedial work can be costly. Recently we have been asked to look into and report on a number of structures which for one reason or another have been thought to be defective in either design or construction. In most of these cases there are lessons to be learnt. Some emerging possibly because



KEY PLAN



BEAM PLAN



BEAM ELEVATION

more analysis goes into the investigation of the defects than went into the original work?

One such brief was to report on the various defects in construction of a new wharf at Littlehampton, Sussex.

The newly completed gravel handling wharf was constructed in steel sheet piling with a concrete capping beam, which had certain defects, of which the most serious were poorly compacted concrete, cracked construction joints and misalignment.

The owners of the wharf, John Heaver (Concrete) Ltd., asked us to investigate and report on the defects in detail, assess their importance with special reference to the durability of the concrete capping beam and supervise any recommended remedial work, including the realignment of the capping beam. We were not asked to comment on the design of the wharf.

The recommendations include the construction of a new reinforced concrete top and external face to the capping beam, fully bonded with the existing sound material.

The general contractor, who is located in

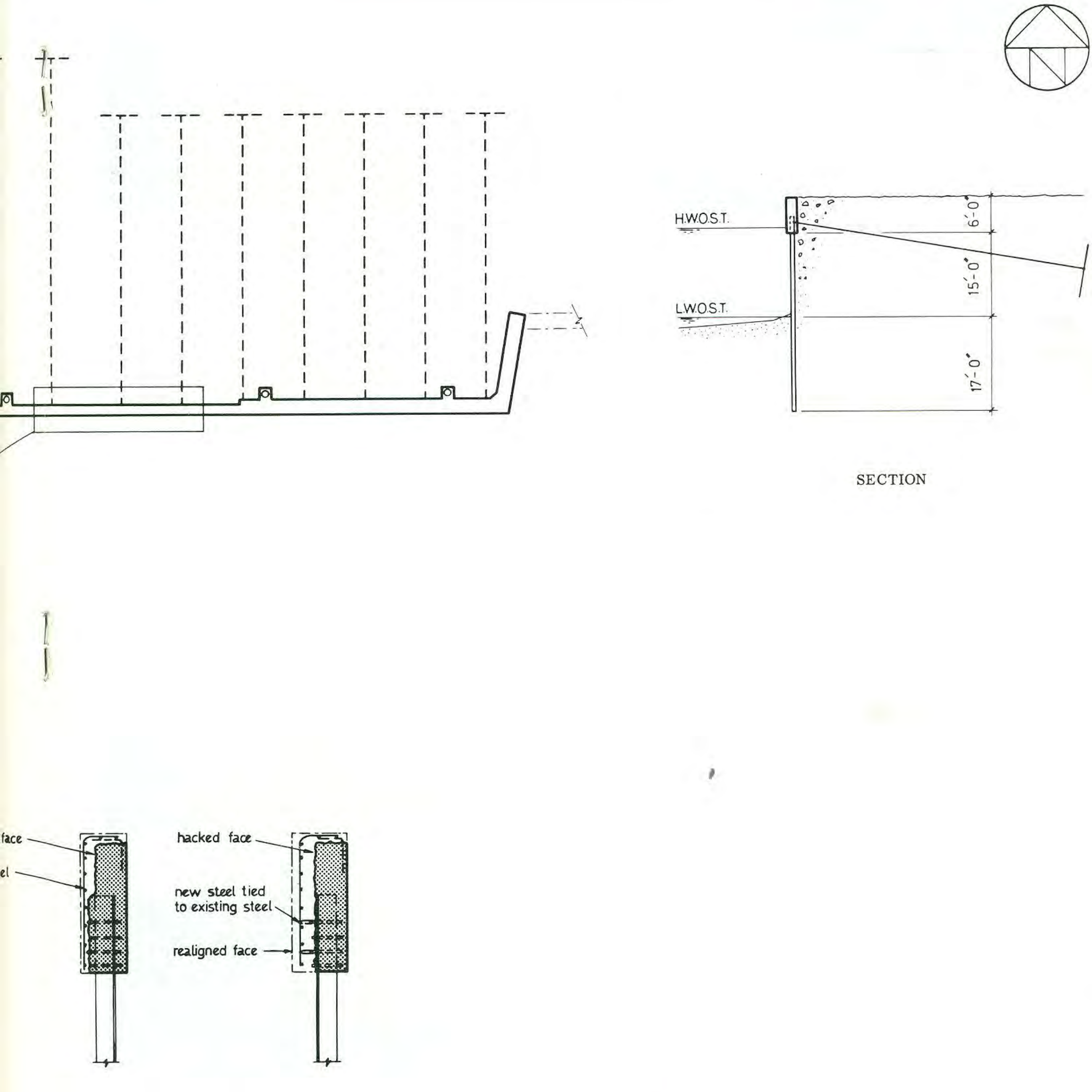


Fig. 1
Key plan and section; beam remedial work details
Illustrator: Marion Raine

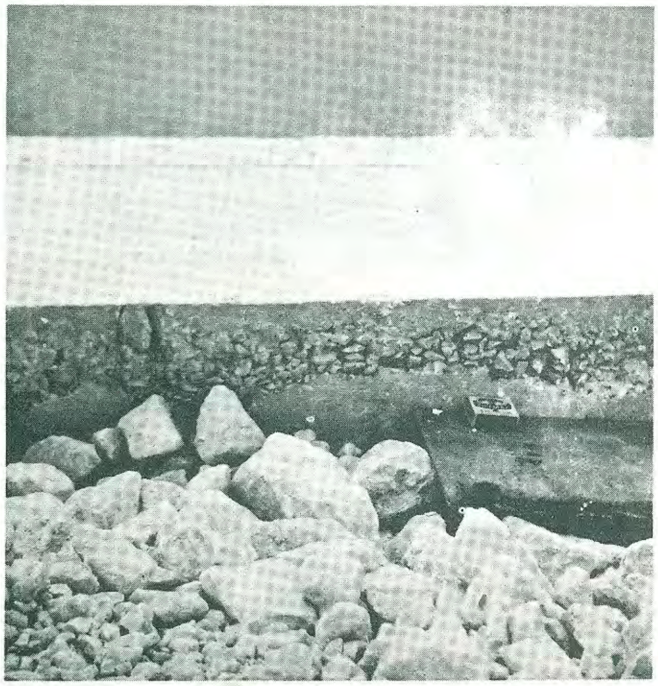


Fig.2
Honeycombed concrete at top of capping beam

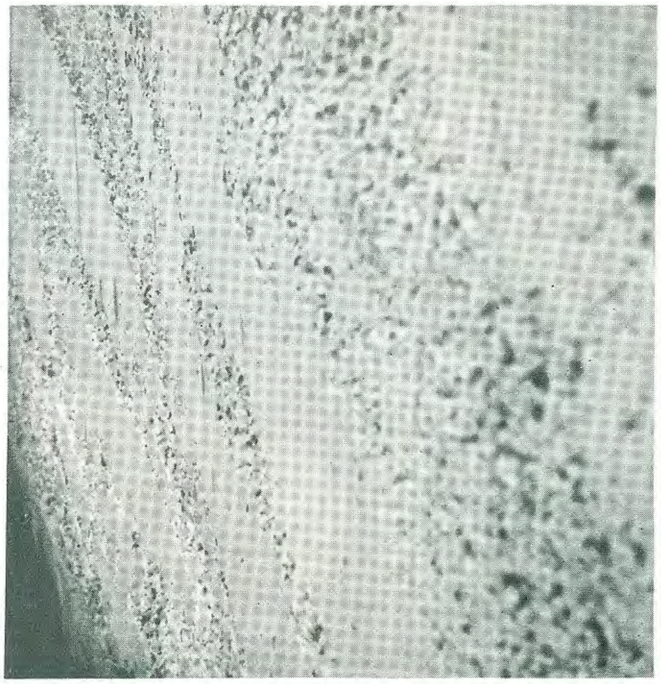


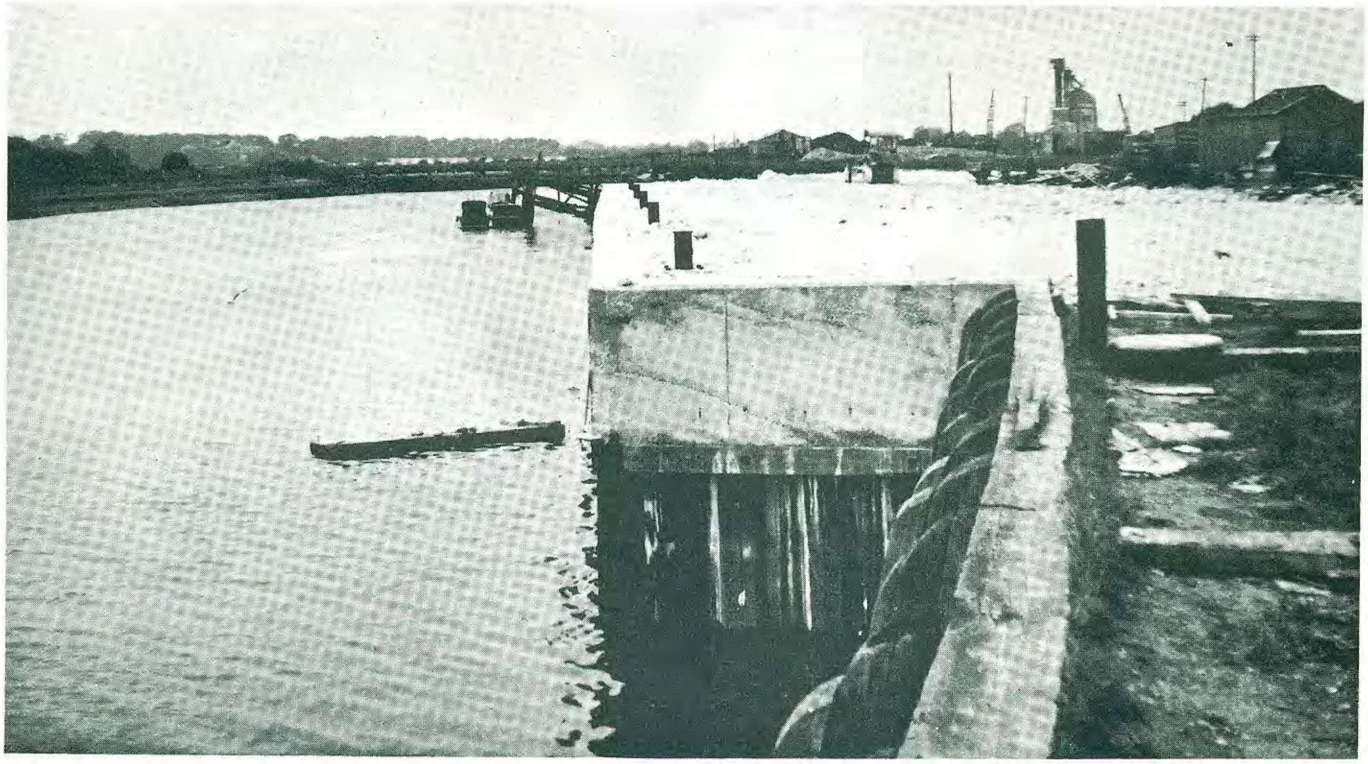
Fig.3
Honeycombed vertical face of capping beam

Portsmouth, constructed the wharf to the design of their consulting engineers.

THE WHARF
The wharf construction consists of a line of 35 ft. long steel sheet piles driven down into the foreshore bed and protruding approximately 18 ft. above bed level with a 6 ft. deep reinforced concrete capping beam. The composite structure is tied back to anchors by means of mild steel rods. The overall length of the wharf face is 240 ft. with short returns at each end. The area behind the sheet piling is backfilled to the top of the capping beam with consolidated chalk. High water of ordinary spring tides submerges the lower foot of the capping beam.

HONEYCOMBING OF BEAM
A fifth of the length of the reinforced concrete beam had been cast approximately 4 in. below the required level. This last 4 in., poured after the main beam had hardened, was not consolidated, nor had the previous face been hacked and cleaned to receive it. The result was a porous joint with 4 in. of honeycombed concrete on top (fig. 2). Further honeycombing occurred on both vertical faces of the reinforced concrete beam for a half of the remaining length of the wharf (figs. 3 and 4). This porous concrete extended to the full depth of the cover.

Fig.4
End elevation of wharf looking west



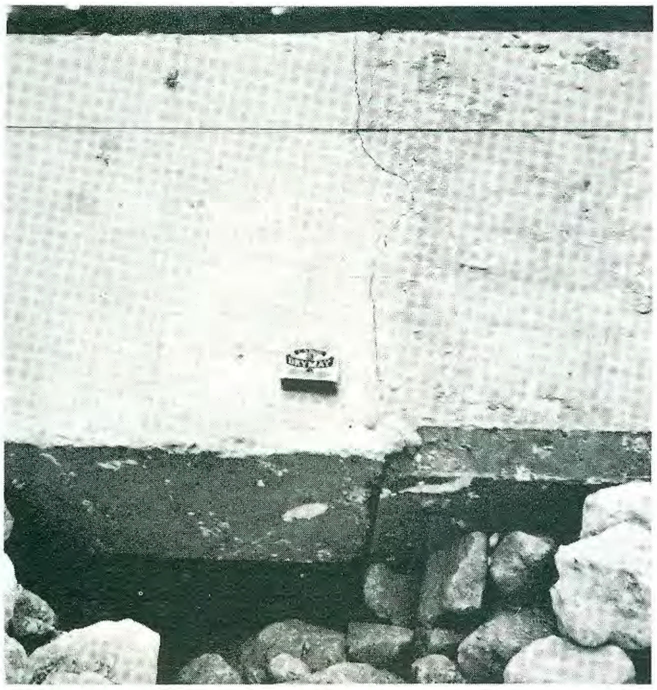


Fig. 5
Typical construction joint crack

CRACKS IN BEAM
The reinforced concrete capping beam had five significant cracks extending at least half the depth of both vertical faces and across the top, a typical one being shown in figure 5. The cracks were due to one, or a combination, of the following reasons - lack of preparation of the vertical surface of the previous pour, poor consolidation of the subsequent pour, shrinkage and the badly chosen position for a construction joint which coincided with a tie rod anchorage.

MISALIGNMENT OF BEAM
The reinforced concrete beam was not cast straight in plan. A kink in the beam occurred near the centre of its length, making this point project 5 in. from a straight line joining the east and west ends of the beam (fig. 4).

ASSESSMENT OF DEFECTS
Two of the prime requirements for the durability of reinforced concrete in or near the sea are:

1. That the reinforcing steel is adequately protected from the atmosphere and the water by a sufficient cover of dense concrete.

2. That the concrete is highly resistant to mechanical and chemical attack by being made of inert aggregates in a fully compacted, dense and sufficiently rich mix, finished hard and smooth. If water can penetrate the concrete because of poor compaction, cracks, or for any other reason, the life of a marine structure will be considerably shortened.

For these reasons the honeycombed and cracked concrete was not acceptable.

The beam was also out of straight by an amount which greatly exceeded the working tolerance of $\pm \frac{1}{2}$ in. which would be reasonable for this type of work.

REMEDIAL WORK
All honeycombed concrete, both on the top and the vertical faces of the capping beam was broken out to the sound concrete below and renewed.
For the length of the beam which was misaligned, the honeycombed areas on the outside face were hacked back to solid concrete. The areas which were not honeycombed were also hacked to give a roughened face thus ensuring a good key for the new facing concrete. Along this complete length the existing reinforcing steel was exposed at least every 2 ft. to enable new reinforcement, which was part of the new facing concrete, to be tied back to it (fig. 1). The outside vertical face and top section were reinforced with longitudinal steel and L stirrups, both being tied to the existing steel at intervals. New facing and topping concrete was cast lining up with the outside face of the straight section, thereby giving a true line throughout the whole length of the beam. The new facing extended for the full depth of 6 ft. of the capping beam with a minimum thickness of 8 in. to permit proper consolidation.
Where a crack occurred in that part of the capping beam where the vertical face was not honeycombed, the concrete was cut out to the full extent of the crack together with any unsound concrete on either side. This section was then recast. During this operation the beam was unable to take any load at that point. Therefore, rather than propping it, which might have proved difficult, the backfill was temporarily removed for 10 ft. on either side. Where cracks occurred in the honeycombed section of the vertical face of the beam, or in the misaligned section, they were automatically rectified by the work carried out in the remedial work for these latter defects.

TOLERANCES
The working tolerances on overall alignment of the beam face, including the remedial work, were specified as $\pm \frac{1}{2}$ in. from datum. As the remedial work on the capping beam was carried out under easier conditions than those existing during the construction of the original beam, the working tolerances on all faces of the new concrete work were specified as $\pm \frac{1}{4}$ in. from datum, with no visible line changing direction by more than $\frac{1}{4}$ in. in 50 ft.

MATERIALS
The concrete used for the remedial work was an approved mix with a minimum crushing strength of 5000 p.s.i. at 28 days. The cement/aggregate ratio was approximately 1:4 by weight with a water/cement ratio of approximately 0.46.
Sulphate-resisting Portland cement should be used for concrete construction in marine conditions. However, as ordinary Portland cement had been used in the original construction it was used in the remedial work to ensure a uniform life and colour throughout the complete length of the structure. The reinforcing steel was mild steel complying with B.S. 785.

WORKMANSHIP
It was stressed that the remedial work should be carried out to the highest standards of workmanship throughout, in accordance with CP.114 (section V) and, in particular, all construction joints should be in accordance with clause 501 of that Code. It was pointed out that special care should be taken in preparing existing concrete faces prior to casting new concrete against them and in compaction of the new concrete, particularly where, because of thin sections, external vibration would have to be employed.
The length of the facing pours was kept down to 12 ft. in order to minimize the effect of shrinkage and also because it conveniently fitted in with the shutter sizes used. Where defective concrete was cut out, the period during which the steel was exposed was kept to an absolute minimum, usually 3 days in order to avoid corrosion.

CONCLUSIONS
The remedial work was carried out by the main contractor who constructed the original wharf. In general the workmanship and tolerances specified were achieved.
Nine months after completion of the remedial work, the only defects in the reinforced concrete capping beam are three superficial shrinkage hair cracks across the top and one slightly larger crack across the top and a quarter way down the sea face of the beam. None of these should have a detrimental effect on the function or life of the structure.

The Meeting House at the University of Sussex

Keith Ranawake

INTRODUCTION AND GENERAL DESCRIPTION

The Meeting House is a two-storey building with an interdenominational chapel for 350 people at first-floor level and a recital room together with offices at ground-floor level. It is circular in plan with a diameter of 80 ft. and is surrounded at ground level by a pool over which there are three points of access spaced at 120°. There is a walkway beside the pool formed by setting back the enclosed ground floor area from the outside face of the building. Twelve brick-faced piers projecting into the pool indicate the main points of support to the superstructure.

The wall enclosing the chapel is 2 ft. thick and is composed of a honeycomb of concrete blocks. These are profiled to take coloured glass window panels set alternately forward and backward of the block face to achieve deep reveals both internally and externally. The glass panels are fixed directly into the concrete blocks.

The roof is a reinforced concrete shell of conical form with a large elliptical opening at the top facing south and bounded by an edge beam. The axis of the elliptical cylinder formed by the edge beam points towards the altar area. The clear opening is 19 ft. x 11 ft. and is covered by $\frac{1}{2}$ in. 'Armourplate' glass with a pattern of coloured glass which is resin-bonded to the soffit of the tempered glass. This elliptical window symbolizes the eye of God.

THE STRUCTURE

The main elements in the superstructure are the shell roof, the honeycomb concrete block wall, the suspended slab and supporting elements.

The shell roof is conical in form, transforming from a horizontal circular base at the bottom to an inclined elliptical opening at the top. The lower ring beam sits on 60 rubber pads with a low shear stiffness. This permits the ring beam to deform horizontally relative to the wall without throwing too great a lateral load onto the wall. Steel dowels cast into the blocks and adjacent to the bearings, project into sleeves formed in the ring beam. These dowels were grouted in three months after the shell was unpropped. This provision has been made because there is no information available regarding the durability of the rubber bearings over a long period of time. Grouting the dowels in after a period of three months ensures that the major part of the deformations due to dead load occur before the beam has a positive shear connection with the wall. The deformations due to live load are small and those due to temperature variations are also small and limited because the shell itself is well insulated.

The honeycomb wall acts as a brittle circular cylinder restrained at the top and bottom by the circular edge beams to the shell and the suspended slab. The final stability of the wall is dependent on the compression induced in it by the dead weight of the roof. There is a factor of at least two against cracks forming on the inside face of the wall between the blocks when the lower ring beam to the shell deforms outwards. The critical section of the wall is approximately one third of the way down from the top.

The reinforced concrete suspended slab is 1 ft. 3 in. thick and 80 ft. in diameter supported by a central column and an edge beam around the periphery. Two 9 in. reinforced concrete walls provide additional support to the slab where the stair wells occur. Columns and walls extend 10 ft. below ground level to mass concrete foundations bearing on chalk.

Meeting House without the lid on
(photo: E. Mark)

CONSTRUCTION OF THE BLOCK WALL

The blocks are of unreinforced concrete cast with horizontally boardmarked finish and were bedded on each other using a 1:3 mortar bed. The main construction problem was to ensure that the bed thickness was maintained at 3/8 in. and that the contact area between the blocks and the mortar bed extended over the whole block overlap. Achieving this would also ensure that no water seepage through the mortar bed occurred. The placing of the blocks was difficult because of these requirements and also because the blocks were rectangular in plan and a parallelogram in cross-section.

These problems were solved by using horizontal and vertical templates with frequent use of a plumb bob and radial measurements to the centre. There are 14 courses of 30 blocks and the contractors achieved a rate of one course per day. A higher rate could have been reached except that it was thought necessary to ensure that each mortar bed had set before the next course was placed.

ANALYSIS AND DESIGN OF THE SHELL ROOF

The shell roof, as originally conceived by Sir Basil Spence, consisted of two coaxial opposed half-cones with one apex twice the height of the other and connected along the boundaries by a concrete truss. Fortunately, Ove Arup met Sir Basil at the appropriate time and they arrived at the conclusion that this particular form was structurally not very desirable nor economic and there was a distinct possibility that it could not be made to work. The final form then developed from Ove Arup's suggestion that straight line generators could be rotated in space with one end on the base circle, in such a way as to form a suitable opening for the window. A string model which could easily be made to change shape and also have different openings at the top was used to decide on the final form and proportions of the whole system.

The vertical live load was taken according to CP3 Chapter V as 14 lb per sq.ft. The data used for the wind loading was obtained from the National Physical Laboratory Aero Report 1009 which gave internal and external pressure distributions obtained with horizontal wind flow on a truncated right cone. The internal pressure distribution contributes a significant component to the total loading on the roof. The NPL report is based on an investigation which predicted the wind pressures on Liverpool Cathedral during and after construction, and indicates that the pressures recommended in CP3 Chapter V are too high for this particular form.

Two different methods of analysis were used, one was a membrane solution which R.S. Jenkins developed from previous work he had done on the Shire Hall shell at Shrewsbury, the other was the solution of an equivalent space frame using a general computer programme. The results obtained from this second analysis gave some indication of the bending moments in the shell which, as one would hope, turned out to be very small. It also gave the moments, forces and deformations in the edge beams. The results of both these analyses will be compared at a later date.

The results obtained from the equivalent frame analysis appear to be of sufficient accuracy to suggest that prior to the proper analysis of a shell of peculiar or difficult form, a preliminary design could be investigated by using an equivalent space frame to determine quickly and at no great cost whether the proposed form of shell is practicable. The detailed analysis could then follow.

CONSTRUCTION OF THE SHELL ROOF

The first possibility considered was a reinforced concrete shell. A laminated timber shell was also considered and rejected because of the cost. Timber shuttering for the concrete was not an economic proposition because the form of the shell made it certain that there could be very little re-use of the timber.

The solution eventually adopted was to use light gauge 'Hy-Rib' expanded metal sheets as permanent shuttering spanning not more than 2 ft. 6 in. between 8 in. x 2 in. timber joists which followed the generator lines. Extensive re-use of the joists at a later date was possible as long lengths were used. The light gauge 'Hy-Rib' is reasonably priced.

A mock-up of the shell inclined at the steepest slope of 41° was made (minimum slope = 34°). Two concrete mixes were experimented with and a grade 5000 mix with a maximum aggregate size of $\frac{1}{2}$ in. which was hand tamped was selected. Some difficulty was envisaged in the actual construction but in fact no major problems were encountered. The contractors took 2 weeks (10 working days) to concrete 600 sq.yds. of the shell with the edge beams already in position. The nett structural thickness of the shell is $2\frac{1}{2}$ in. The finishes to the shell are $\frac{3}{4}$ in. of expanded polyurethane of a specified density with copper sheeting on the outer surface and 1 in. of 'Pyrok' sprayed asbestos on the inner surface. The sprayed asbestos was placed in segments of varying density and the surface alternates between being acoustically absorbent or acoustically reflecting.

BREAKDOWN OF COSTS FOR THE ROOF

Total cost	£17,040
Structural cost	
1. Lower ring beam	£4,192
2. Shell	£2,572
3. Upper ring beam	£ 806
	£7,570
Finishes	
1. Copper, insulation, etc.	£7,625
2. Glazing to the elliptical window	£1,845
	£9,470
Approximate areas	
Surface area of shell	= 6000 sq. ft.
Glazed area	= 700 sq. ft.
Total plan area	= 5200 sq. ft.

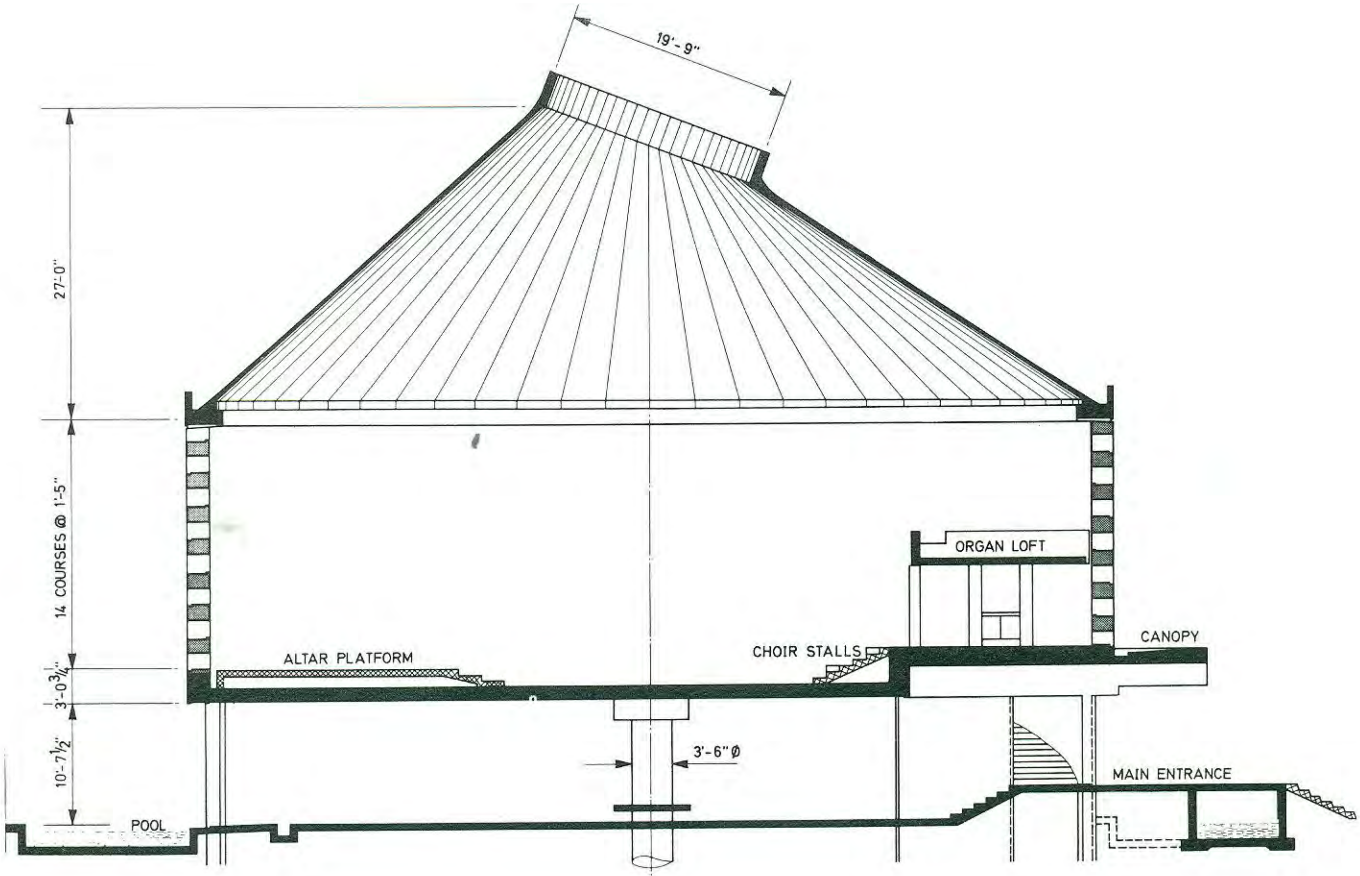
If the figures for the structural cost are examined, it can be seen that the unit costs for the shell itself are 8s. 6d. per sq.ft. based on the true surface area and 10s. per sq. ft. based on the plan area. These figures appear to be reasonably economic for shell construction and seem to indicate, that the method of construction adopted is particularly suited to a shell whose surface is defined by straight line generators.

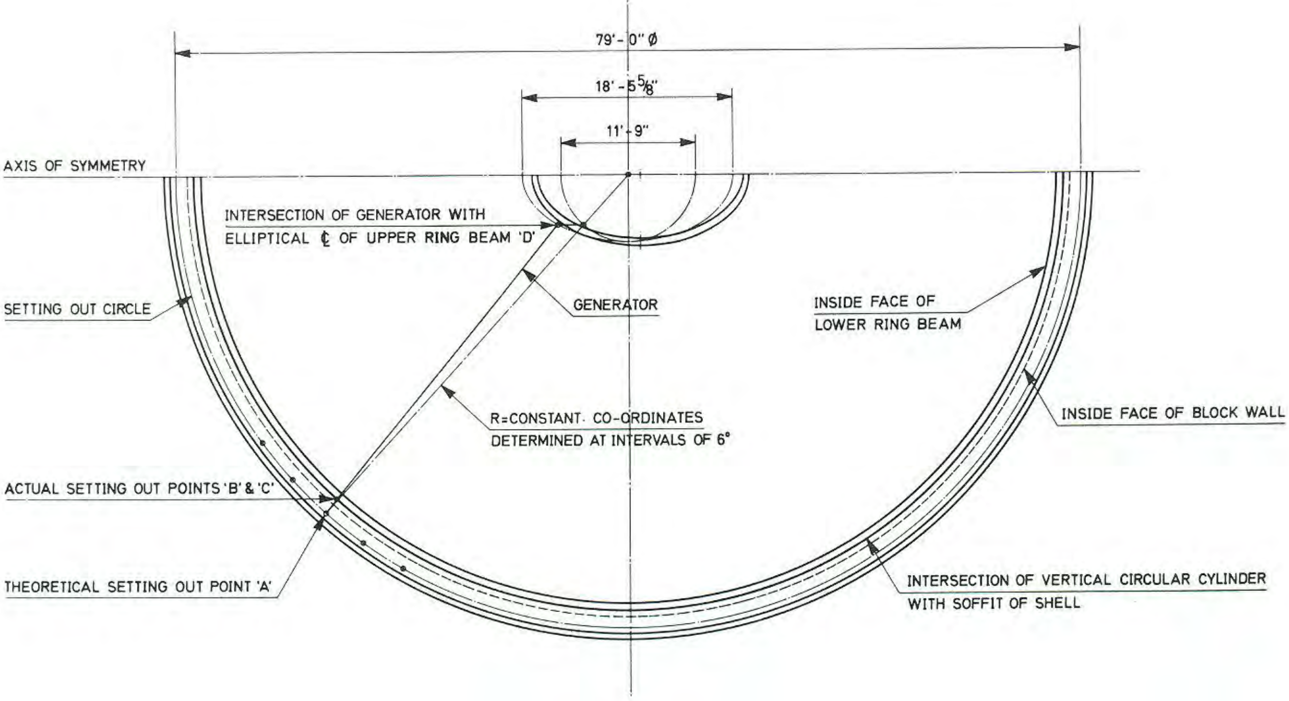
The unit costs for the shell ring beams are £18 per ft. run and £17 per ft. run for the upper and lower beams respectively. These figures are very high and can be attributed to the constuctional difficulties encountered in each case. The cost of the finishes speaks for itself.

The total cost of the building is approximately £85,000 of which approximately £35,000 is for the structure.

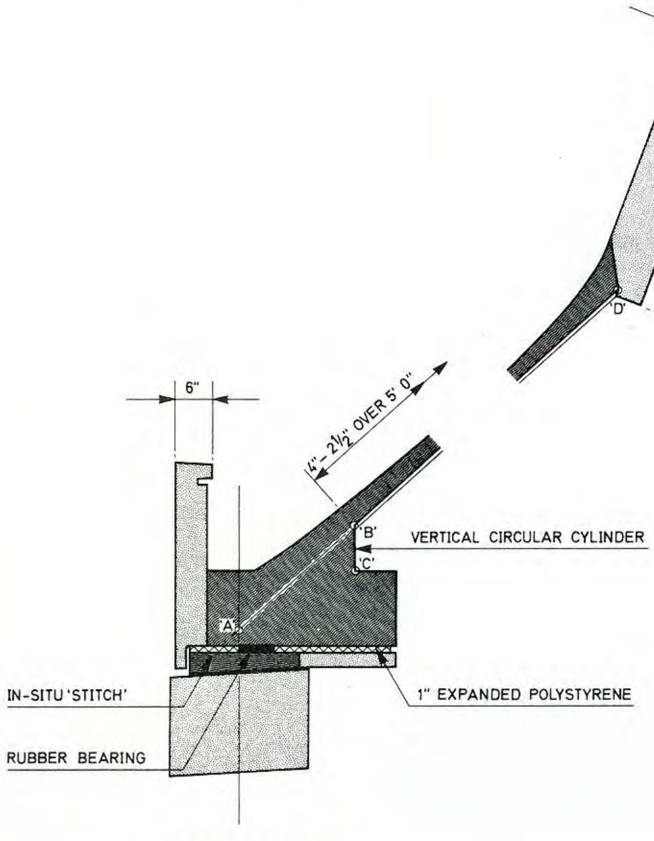
This project came into the office in February 1964. In the year that followed major changes of form took place and the final design commenced early in 1965. Work started on site in October 1965 and the structure was completed in April 1966. The scheduled date of completion is October 1966.

View of completed concrete shell
(photo: E. Mark)

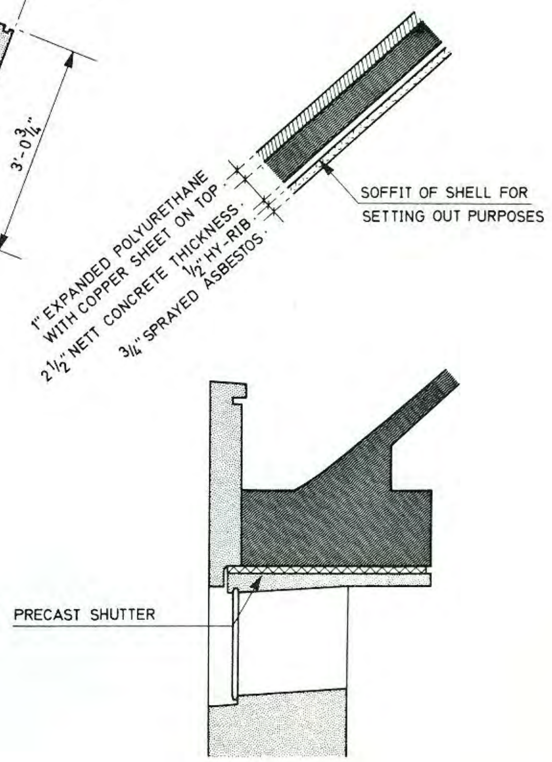




Half plan of roof



SECTION B-B



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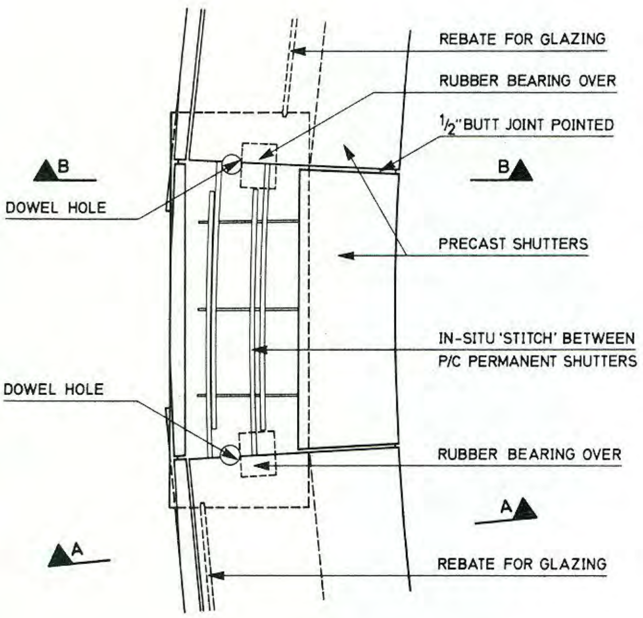
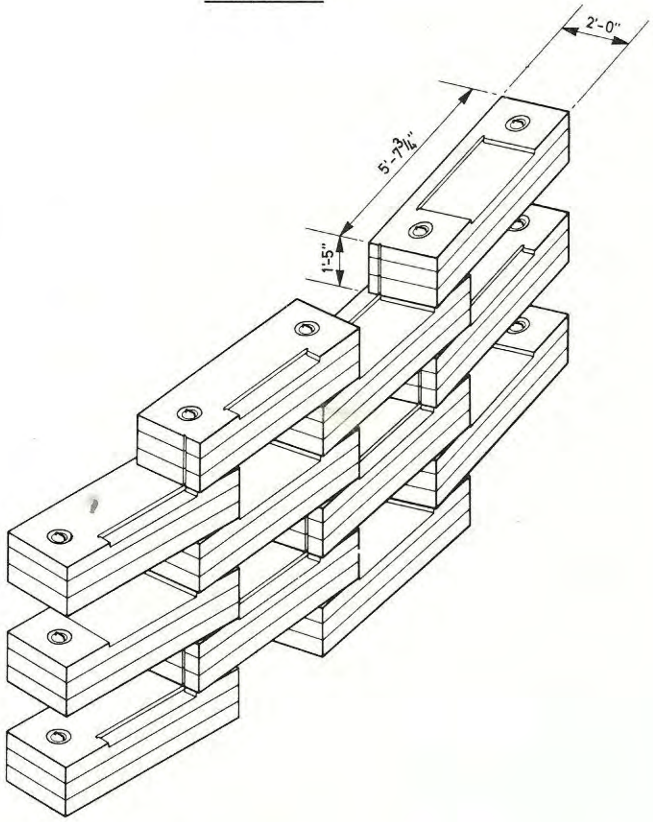


PLAN OVER WINDOWS



ISOMETRIC OF BLOCKWORK

Miscellaneous details

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