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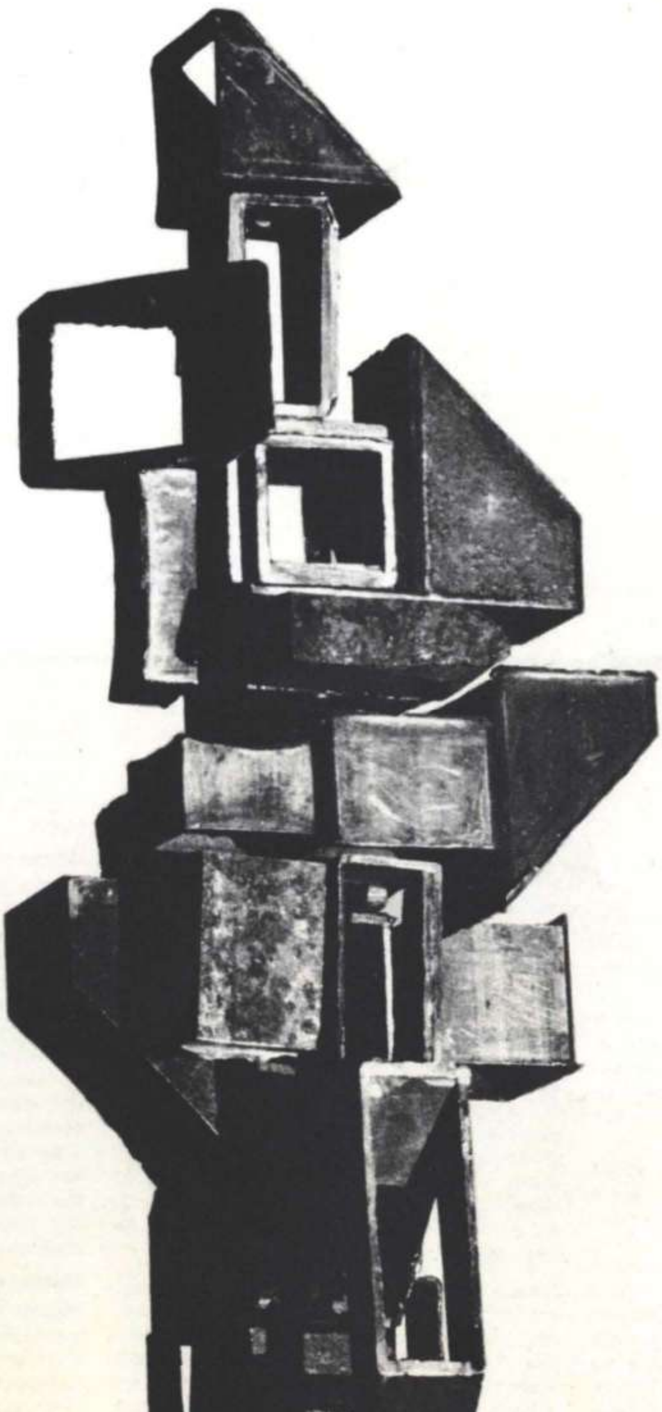

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Front and back covers: Two aspects of a sculpture by Monty Sack. Owned by Peter Dunican. Photo: John Donat.

Heavy concrete systems in low-cost housing

V. J. A. Kemp

The term industrialized building is still new to the industry as a whole and provokes many strong reactions ranging from extreme distrust to dedicated enthusiasm.

Whichever view one takes, it is apparent that this type of building demands the ultimate in collaboration between the various designers—and I use the term designers deliberately so as to include the constructors as well as the so-called professionals

It would be unreasonable to expect any single member of the design team to be an efficient and effective Jack of all Trades, but it is most desirable that he should have a working knowledge of all the factors which go to make up a successful building—and it is vitally necessary for him to have a sympathetic understanding of the problems in the areas outside his particular speciality. Perhaps in no other section of the industry are the design problems so inter-related, and very few problems can be separated out and solved by any single member of the design team in isolation.

Timely involvement

The first problem is that of early involvement in the scheme. Although the principle of setting up the design team at an early stage is becoming far more prevalent, one is still faced with the situation from time to time when the architect presents a scheme in the final stages of development which he is committed to building with industrialized methods. Eventually one does find a possible system but all too often that system has to be considerably bent —if not totally distorted—to achieve the architect's aims.

This can only be done at a price, and in these circumstances, one is often faced with an agonising re-appraisal when the final price emerges. By this time, the scheme has passed the point of no return, sensible basic rationalisations would inevitably set off a chain reaction of consequences too numerous to contemplate. One is left with the alternatives of reducing standards and quality—whether it be in terms of performance specification, or aesthetics—or starting again.

Ideally, the engineer should participate from sketch plan stage. Initially he needs to consider the basic vertical and horizontal stabilities of the proposed structure and to relate them to whether the scheme is suitable for system building in the full sense, or whether it is better achieved in rationalised traditional forms of construction. In passing, it should be noted that the geographical location of the scheme, the probable work load in the area at the time when construction is scheduled, the type, quantity and quality of the local labour force are all additional factors that used to be considered, and evaluated if possible at this stage.

Cross-walls

In principle, most of the heavy concrete panel systems have much in common from the structural viewpoint, in that they are generally cross-wall structures with non-loadbearing external walls. They differ quite considerably in detail and each system has its individual advantages and disadvantages. Some have developed the ability to provide a certain amount of continuity at the joints. Others have the means to provide for limited vertical tensions. In general it is not a practicable proposition to resist forces and moments of any magnitude, and the engineer must guide the architect into producing a scheme where the structural layout provides the necessary stability within itself.

Rationalisation

Having satisfied himself that the basic structural needs of the building have been met, the engineer must turn his attention to the detailed embodiment of the building and here we start running into problems which are secondary in the structural sense, but of vital importance for a successful industrialized scheme.

Rationalisation is a basic and essential prerequisite of successful industrialization. In general, and in isolation from the detailed requirements of any particular system, it is possible for a design team to produce a simple, repetitive design that should be capable of being built by a number of different system builders. Unfortunately, each individual system has its own particular disciplines and limitations, and in producing such a scheme, one is inevitably building into it a number of restrictions that ultimately will not exist, and which may well have had a profound effect on the basic concept of the scheme.

Undoubtedly, the best chance of success lies with the design team who set off with the intention of using one particular system. They have known limitations and disciplines with which to contend, but they also have the chance of exploiting the existing possiblities and developing the potential within realistic limits.

In this situation the engineer is able to proceed with confidence. The implications of the production process can be fully assessed, the capacity of the handling equipment is known and the scheme can be broken down into its constituent parts equating structural demand with constructional facility.

Architect as leader

On the face of it, this argument could logically be extended to promote the principle of package dealing, particularly so if sheer productivity were our only yardstick. However, if the needs of the community are properly to be met, whilst still retaining an acceptable level of environment, then I am personally committed to the principle of a strong, client-sponsored development team controlling the development, led by the architect and working in close asssociation with the production side of the industry. In other words—a negotiated contract, and I am sure that the means exist properly to control such an exercise, to ensure that cost limits are not exceeded and above all to ensure value for money spent.

The engineer's task

When an engineer engages in industrialized building to the extent that he provides full working details from which the scheme will be built, he immediately becomes aware of the enormous additional workload that becomes necessary in the design office.

Traditionally the engineer's task is to produce the design, the calculations to support it, the dimensioned outline drawings, the reinforcement layouts and usually the bending schedules. In doing so he will make due provision for any major holes that are structurally significant but the general problem of co-ordinating services requirements including electrics, fixing blocks etc. is left to the contractor's site organisation—as are the fundamental questions of construction methods and sequences.

The main aims of industrialization are to produce more with the existing site labour force—to reduce the skilled labour content to the minimum and to make better and more extensive use of semi-skilled and unskilled labour. And yet, at the same time, we are presenting the construction side of the industry with a complex problem requiring more sophisticated methods than are generally available.

In these circumstances it is clear that the design team, and more particularly the engineer, must take over the role of co-ordinator. The man on the site must be given clear and precise instructions down to the last detail as to what is required of him, leaving him free to exercise his talents in the field of organization and management, which are the very essence of industrialization. We are moving away from the traditional crafts, although in the process we are tending to produce new ones at the factory-floor level.

It is no longer good enough to tell the man producing a precast concrete element its size and reinforcement content, leaving the rest to him. He must also be told exactly what electrical fittings and fixing blocks are required and where they should be positioned. He must be told precisely what lifting devices, levelling sockets and propping points are necessary and exactly how and where they must be fixed.

Under current development procedures, this sort of detailed information is usually the last to be made available, and yet the structural drawings are required very early in the construction programme, which leaves the engineer with the production problem in his drawing office, in that a very great deal of detailed information has to be set down accurately in a very short time. The need for realistic pre-contract programmes, rigidly adhered to, cannot be over-emphasized.

During this detailed planning stage, the engineer must investigate the structural implications of early striking, lifting and stacking on the individual element. It is quite possible for the initial, temporary, condition to produce a worse structural situation than when the unit is finally fixed in position in the structure. He must also consider how the building is to be constructed and in what sequence the individual components are to be erected. Failure to do this can easily result in local instability or, at best, finding that the last unit cannot be erected owing to the physical constraints of the surrounding structure.

Having dealt with the precast elements themselves, the engineer must now produce drawings showing how the pieces fit together. With a basic grid established, and the elements dimensioned to suit, the layout drawings need only to be diagrammatic, but they must be presented in sufficient detail to ensure that no ambiguity can arise in the erection stage.

They must contain certain basic information such as erection sequence and precise information as to numbers of temporary props and their exact location. The whole question of propping must be given the most serious consideration. The prop itself must be strong enough to contend with the type of load it may be required to sustain, the fixings at head and toe must similarly be sufficient, simple and foolproof and the angle of inclination to the wall face, in both horizontal and vertical planes, must be such that the full efficiency of the prop is not seriously impaired. With all but the smallest elements, two props should be used per wall as a general principle.

A wall, when erected, presents a large face area relative to its self-weight. In the temporary state, it is extremely vulnerable to sudden gusts of wind or to an accidental blow from another unit suspended from the crane hook. The failure by overturning of any one such unit could well cause other units or other props to fail and the situation could rapidly escalate into total and catastrophic collapse of the structure. With these consequences in mind, no short cuts should be contemplated, no economies should be effected in this area without the most rigorous examination.

In all of this, it must be remembered that the structure, even a relatively costly industrialized structure, only represents something rather less than 40% of the total building cost. Successful industrialization affects the whole of the building process, and even the most elegant and sophisticated industrialized structure can really be counted as a failure in these terms if the remainder of the process is not planned and executed in similar manner.

Heavy concrete panel systems will undoubtedly continue to have a large and important role to play in the future. It is possible, however, that the main impact of industrialization may well come in less spectacular, but no less important form.

In the industry as a whole, mechanisation is under way—and still has a long way to go—but we have hardly commenced on the basic problem of rationalizing the building process in all its forms from the design concept to the final hand-over.

O & M

I, too, look forward to the time when industrialized building will become *normal* building and I think it matters little whether our construction medium is steel, concrete, plastic or green cheese. Above all, the nub of the problem lies in organization and management, and in this too the engineer will participate, even though he may not be using his slide rule and computer in the traditional way.

Instability of large panel construction

D. P. Kerns

A traditional reinforced concrete building can generally be considered to be a monolithic structure, that is to say, the individual structural components may be connected together in such a way that direct forces and bending moments can easily and naturally be transmitted. In this way, the individual components can be made to act as frameworks which provide structural rigidity and stability as well as the ability to indulge in irregular and minimal structural arrangements, if required.

With the majority of available heavy concrete panel systems, these facilities no longer apply. It is difficult to connect large pieces of precast concrete together and achieve the same degree of cohesion and continuity as with a traditional in situ structure. Theoretically, it is structurally possible, for instance, by using prestressing techniques, to achieve the same result, but it complicates the building process and at this point in time is not economically viable. Essentially, the industrialized structure is a gravity structure dependent on its mass

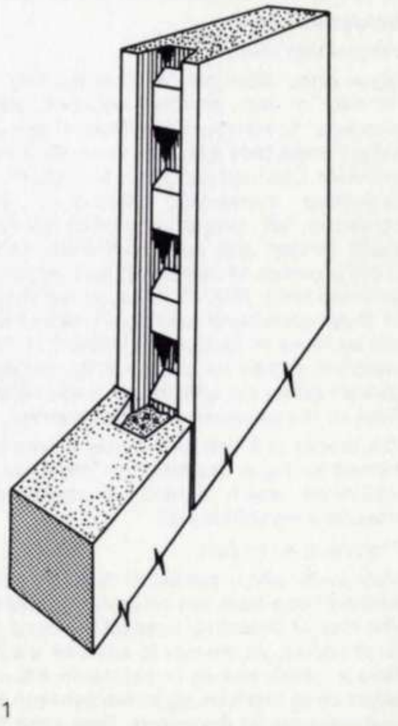

Fig. 1

and the distribution of its mass for its overall stability.

Wind stability

Horizontal design loads

Besides the normal wind pressures for which buildings are designed, is it necessary to design for any additional horizontal forces? Due to the articulated nature of the structural joints and the possible effects of earth tremors, differential settlement and building out of plumb, it is felt that an additional force equivalent to ½% of the total dead and live load of the building acting uniformly and horizontally should be allowed for. It is understood that this design requirement will eventually be applied to all types of building. The investigation into the effects of a diagonal wind on to the unclad building is not necessary with the cladding being erected simultaneously with the structure and finishing trades close behind.

Low-and medium-rise blocks

Most architects prefer to plan these buildings with cross-walls in one direction only. Therefore a stability problem in the longitudinal direction arises. Most systems are unable to deal with this problem without introducing some form of longitudinal wall. Due to the manner in which the slabs span, such walls usually have insufficient load to combat any tensile forces which may arise because of the horizontal load.

There are two ways of overcoming this problem, firstly by using a rather expensive tensioning device which connects one storey height of wall with another and, secondly, by using such a wall as a diaphragm between two cross-walls which are loaded. Some systems provide some means of developing continuity between slabs by the provision of projecting hoop-bars. This is obviously the simplest way of overcoming this problem and causes less headaches for the architect and the system builder.

High-rise blocks

This type of block is usually built to a maximum height of about 25 storeys, so here it is essential that one has walls in two directions and also that these walls be fairly heavily loaded. One should avoid developing any uplift as, when this occurs, any tensioning device which may be considered is usually found to be of such dimensions that it would be quite impracticable. These wind-bracing walls usually have to be tied together by door beams so that some degree of co-action can be developed.

Structural joints

Vertical wall joints

More often than not, a wind-bracing wall consists of two or three separate precast elements. To construct a building of any great height interaction between these elements is essential. One method used is the provision of projecting overlapping hoop-bars which obviously has mould cost implications. A much simpler and equally effective method is the provision of castellated wall edges with an in situ stitch (Fig. 1). Tests on the strength of these castellated joints have been carried out by Harry H. Stanger for Wates Ltd. These were not entirely conclusive as the testing rig yielded before the ultimate load was reached. Even so, the results were quite impressive.

The braces in a belt and braces situation are formed by the in situ concrete stitch over the wall-head which contributes considerably towards a monolithic wall.

Projecting hoop-bars

Slab joints which consist of projecting overlapping hoop-bars can solve many problems. The idea of projecting overlapping hoop-bars is, of course, an attempt to simulate the conditions which prevail in normal in situ construction so that framing action between slabs and walls can be developed. Tests have been carried out on this type of joint by Harry H.

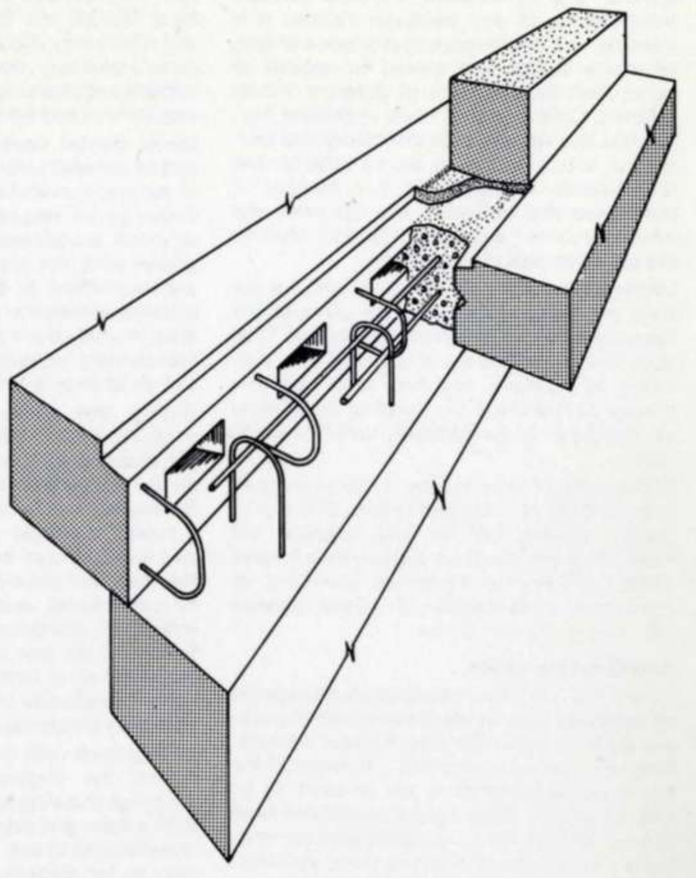

Fig. 2

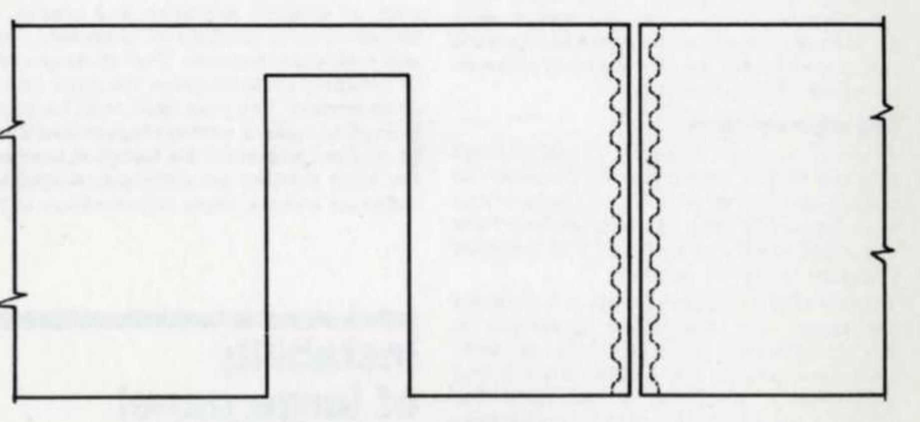

Fig. 3

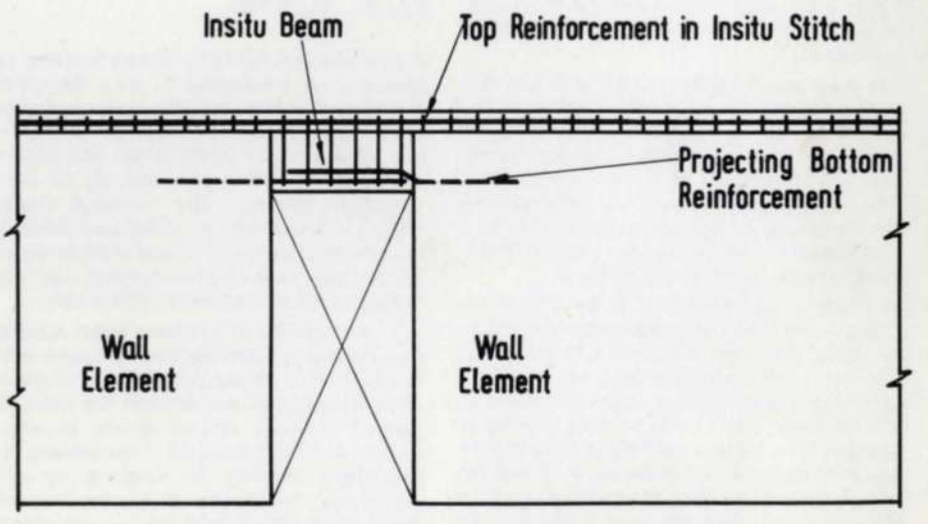

Fig. 4

Stanger for Wates Ltd. (1) and Professor Franz in Karlsruhe (2). From both these tests it has been concluded that the full moment of resistance of the reinforcement can be developed provided a few simple rules are adhered to.

Firstly, the hoops from opposite panels must be fairly adjacent but sufficiently far apart to minimise the possibility of the bars clashing during erection. Secondly, the curvature of the hoop should not exceed an internal radius of seven diameters with an overlapping length measured from edge to edge of 16 diameters for normal concrete grades. It is possible to reduce the curvature but, of course, the length of the straight side of the hoop would have to be increased, together with the overlapping length. It is also felt that the provision of lateral reinforcement inside the hoops is desirable especially towards the edge of such a joint. A typical joint is illustrated in Fig. 2.

Shear wall connections

The simplest and most efficient manner of connecting two shear walls by means of a beam can be done by ensuring that the door opening does not coincide with the end of a wall element. That is to say, the elements should be arranged so that one of the panels has the door opening cast well within its boundaries (Fig. 3). Another method of making this connection which does not require the disciplines just mentioned, is to cast the wall elements with projecting reinforcement and to cast the beam in situ. This is often undesirable as these projecting bars can occupy valuable space in a battery mould. Also the in situ concrete is never of the same quality finish as the precast concrete. The top reinforcement, of course, would be in the in situ stitch (Fig. 4). A third method which has been tried is by providing a much shorter threaded projecting bar from each wall unit and connecting the two together by means of a *MacAlloy* bar. The only problem here is the degree of accuracy required in both casting these units and levelling during erection. Site welding has been considered but the quality of such often falls short of the necessary minimum standards.

Finally, a method on trial at the moment is providing pockets in the two wall elements to be connected and using a precast concrete beam bedded on mortar in these two pockets (Fig. 5). It is a very similar principle to building a cantilever beam into brickwork. The length of beam necessary to be built into these pockets has to be decided. At the moment a rather conservative approach is being employed, i.e. the length of embedment is made equal to the bond length of the reinforcement in the beam. Further investigation is proving that this can be substantially reduced.

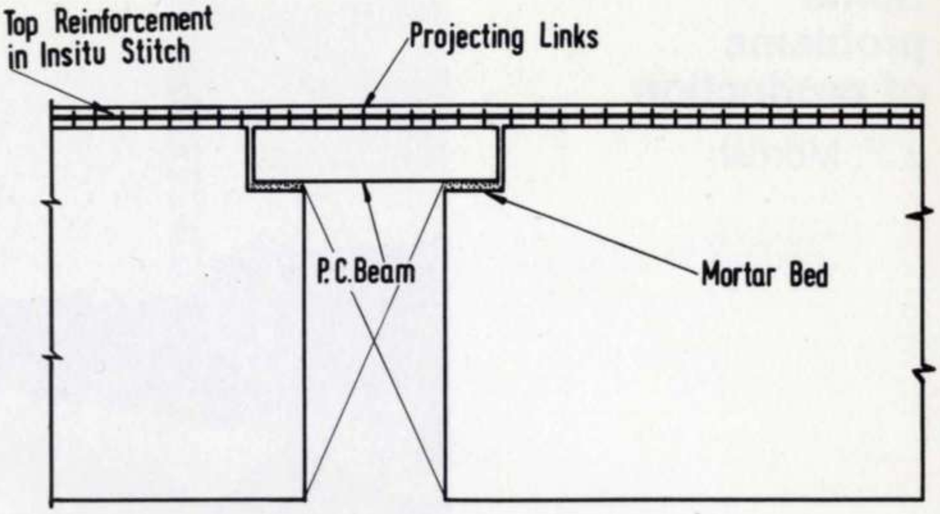

Fig. 5

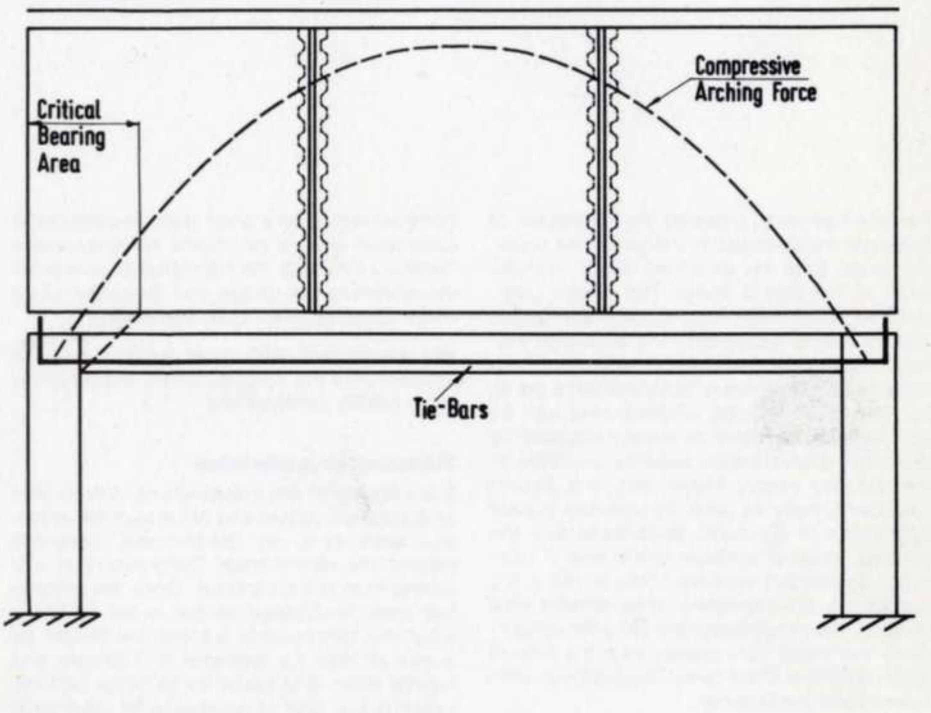

Fig. 6

Slender sub-structures

In Europe the majority of multi-storey residential blocks continue down to ground level using the same plan form and layout as that on a typical floor. This means that the substructure continues uninterrupted to foundation level. In this country, it is more usual for multi-storey blocks to be repetitive from first floor upwards only and the ground to first storey to be planned for totally different purposes, requiring a different type of structure with a column and beam approach predominating. Needless to say, this part of the structure is invariably cast in situ. One must take care that the compressive stresses and therefore, the elastic shortening in the columns supporting interconnected shear walls are of a similar order, otherwise the relieving effect of the beams connecting these walls can be nullified. The other problem encountered in this transition from a cross-wall structure to a column and beam structure is that associated with deep beams. If a total structure in a block were in situ most engineers would not hesitate to use this approach, but as so often happens, they have a considerable distrust of precast structures. It is felt, however, that provided one has an in situ beam underlying the line of precast elements, the usual deep beam methods of design can be used. All the tension reinforcement is placed in the in situ beam and the arching compression forces are transmitted through the precast walls whether they have vertical joints or not. The horizontal shear forces are transmitted from the in situ beam to the wall bed by virtue of friction. A recommended value for the coefficient of friction between precast concrete and mortar is 0.5(2). In taking this approach, one must however check the local bearing stresses at the base of the first lift to precast walls immediately above the column head. A typical arrangement of this type of structure is illustrated in Fig. 6.

Conclusion

The engineer must obviously change his approach in the design of industrialized structures. The architect is often criticised for designing his building on a traditional basis and afterwards converting the results to a system building. The engineer cannot even contemplate this approach.

He must design and mentally construct the building at the same time, just as he always is supposed to, but often does not. No really new theories are employed in the design but the so-called secondary effects can become critical. Using the right approach there is no reason why the industrialized building should be any more unstable than the traditional one.

Lastly, a warning on the misuse of standard connections. They only work in a given set of circumstances. Nobody needs to be reminded of Aldershot.

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(1) Ove Arup & Partners Consulting Engineers. Job report 2048 Summary of report of bending loading tests of in situ joints between two 'A' type detail as used in the Wates System, carried out by R. H. Harry Stanger, by N.A. Tadlaoui, April 1967.

(2) Franz, G. The Jointing of prefabricated elements with lips (typewritten communication, no date).

Some problems of production

J. F. Morrish

Fig. 1. Wates' site factory at Feltham (Job. No. 1992) (A'Court Photographs Ltd).

He who has yet to undergo the experience of complete involvement in industrialized building must, from his detached perch, wonder what all the fuss is about. The design problems are ridiculously easy since everything is usually simply supported. The drawings present no particular detailing problems once one has decided how many little bits there are to be. The materials and workmanship can be automatically assumed as being controlled by the most sophisticated systems available in the industry today. Motor cars and aircraft have been made for years by infinitely greater extensions of the same techniques and the finished article is a tribute to the skill of relatively unqualified workers. Unfortunately, the operations are repetitive, they require little thought, they must inevitably stifle the imagination and might conceivably smack a little of commercialism since direct involvement with a contractor could occur.

Years of inspired swinging from one continuous beam to another, whilst clad in his carefully tailored institutional cloak, might persuade our perched man that his talents would be better employed if directed toward headier exercises. Some prestressing here, a shell or two there and presto ! a Sydney Opera House. It does not matter which way we choose to go, industrialized building must inevitably come to us all. What a shame we cannot find a more genteel name for what is the logical application of modern resources to the building industry.

Those of us involved will recognise the summary made by our friend. There is much truth contained in it. The processes should be relatively straightforward but—if we may borrow from Mr. Micawber—in short, they are not and, unlike him, we cannot wait for something to turn up, the problems are with us and have to be worked on. Papers are published, international conferences are held and millions of pounds are spent on development but the topics still remain the same and some are outlined below.

It would be impertinent to suggest that the Housing Division has the solutions but at least we within it are becoming more aware and consider this to be a good start. The following comments are by no means comprehensive but should indicate the difficulties encountered in converting our design and drawings into a major industrialized building project.

We are familiar with most systems and the problems are not unique, nor are the solutions more readily forthcoming.

Planning for production

It is a feature of any industrialized scheme that architect, consultant and contractor are mutually dependent on fundamental decisions taken at the earliest stage. The pressure usually comes from the contractor. Once the scheme has been formulated he will want to know what the components are and we should be aware of how he proposes to fabricate and handle them. It is usual for us to be told the capacity and type of cranage to be used since this will affect the maximum weight of the component and the manner in which it is to be lifted. We should be interested in the layout of workshops, stores, offices and factory since these may influence the construction sequence and consequently decide the position and type of joint or continuity device required. A study of Fig. 1 will illustrate most of these points. For our part it is essential that unit outlines, together with connections and jointing details are prepared, and that the total numbers of the various elements are arrived at as soon as possible. Moulds and special equipment can take months to fabricate and the whole casting or manufacturing programme depends on this early information. It may well be that the contractor will have plant available which he wishes to utilise and we must, as far as possible, design around this. The essential thing is to have a free exchange of information and to insist on answers to critical questions. It is at this stage that a bad decision or wrong information given by one party can render the others' work useless.

We have for example detailed a scheme on a basic unit length of 21 ft., only to be met by an irate factory manager and a battery mould that would produce 18 ft. maximum. Somebody had neglected to tell us that this was to be transferred from another site, we had assumed that a 'normal' battery was to be used. Elsewhere the crane could handle $6\frac{3}{4}$ tons, our unit weighed 6 tons, but the chains and new lifting beam weighed nearly 1 ton and this unit has to be placed at the maximum reach of the crane. New lifting gear had to be ordered. These arguments are now dead but the incidents should not happen. Correcting them is a costly and time-consuming business.

Fabrication of units

It is inevitable that our prime consideration is for precast concrete units but the underlying principles which govern presentation of information and control of production of repetitive items remain valid for most materials.

The underlying statement for any prefabricated element suggests a small team of highly skilled craftsmen producing high quality jigs or formers made with great precision to relatively sophisticated and often intricate details. Given a moderate degree of supervision and considerable mechanical assistances, unskilled labour can, with the aid of these formers, produce large numbers of identical units in less time and with greater consistency than teams of skilled men working in situ.

This philosophy is held by the sponsors of the so-called 'closed' systems, the essence of which is to produce only the predetermined range of components and to leave the architect to arrange them as best he can. The real danger lies in the temptation to put the pieces together in the cheapest possible way. Evidence exists all over Europe of the consequences of such a decision. Any system offering greater flexibility in planning immediately falls foul of its own virtues. Rationalisation of structural plans can produce an optimum number of basic forms and the moulds can be produced as before. On even the most complicated high-rise scheme (see Fig. 2) the number of mould changes are relatively few and can be quickly learned by the introduction of a few more semi-skilled men.

The current trend in 5-6 storey blocks with deck

Fig. 2. Preparing fixings in typical six-leaf wall battery (Photo: Gittana Ltd.)

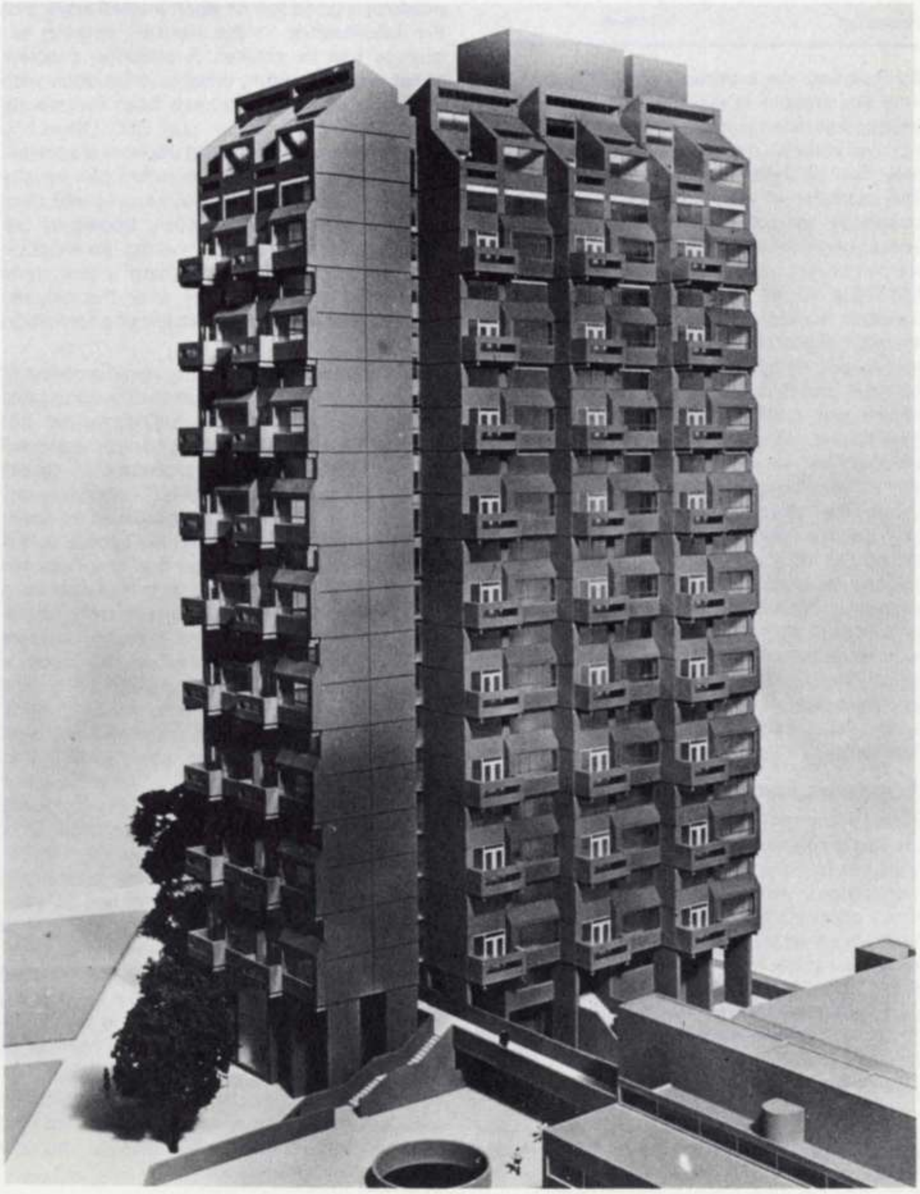


Fig. 3. 23-storey block for Lambeth Borough Council. Wates System (Job. no. 2013) (Photo: Gittana Ltd.)

access, internal stairs, connecting links and varying accommodation, has set problems which have yet to be satisfactorily solved.

We have attempted short-cuts in detailing which, at this stage of site management and drawing office organisation, have sometimes proved to be catastrophic both to ourselves and to the site. On the other hand to attempt to produce the information in a traditional way would involve us in exorbitant costs and anyway the time is never available. As explained earlier, some of all the information is required immediately or as near to it as we can get. Contractors for their part are woefully short of the foremen and supervisors who are capable of exerting the new skills which such careful control demands, and very often pricing does not allow for this control, even if it were available.

Before developing the argument let us examine a typical problem—that of wall 1205 and 1205X. We can see from its reference number that it is about 12 ft. long and has a handed version. It is a structurally ubiquitous animal and can divide bedroom from kitchen, have an external balcony or form the party wall between dwellings. To suit its mood there are six electrical variations, three positions of the heating duct and two positions for fixing blocks. Then of course there is the handed version. So far there is no need for our battery-hand (Fig. 3) to employ his comptometer. Fixing blocks are cheap and, if wastage is accepted, all can be installed. A limited amount of conduit can be duplicated but making good of unused sockets is a charge on the factory. Near-face and far-face is tricky because he is standing between the two and does not know which one he is looking at. By the same token when he turns around to fix to the mould face behind him all left hands automatically become rights. To further his confusion vertical casting of slabs need owe no allegiance to top and bottom as drawn. In planning, the astute mould appreciator has recognised that there is about another 8 ft. of capacity in the leaf and with an eye to the production schedules has carefully organised two or three other small units to be cast as required.

Our non-tradesman can thus be faced with up to 50 changes in the one leaf. Here is one part of the dilemma, do we produce the 50 drawings or does the contractor have a more skilled/expensive operative? Assuming that we can produce the drawings, who will organise their effective distribution on site? We are already experiencing near breakdown in communication with the present numbers being issued. An alternative solution could be to leave the mould unchanged for two or three castings, but the erection sequence may only require such a combination of units every three months. It is easy to fall behind the erection programme in this way and where are the surplus units to be stored? Thus, there are real problems before reinforcement has been fixed or concrete has been poured. Production bonuses without tight control do not lead to careful assembly or stripping of moulds, checking of reinforcement or adherence to water : cement ratios and other specification requirements. Teams of traditional clerks of works have not generally proved to be effective.

Donald Bishop*, in his B.R.S. days, drew up a table of scales of activity which is shown in Table 1.

Table 1 Scale of activity

Method	Cycle	Scale
Site casting	Weekly	1
Temporary factory	Daily	5
Permanent factory		
Steam curing	8 hours	15
Continuous kiln	4 hours	30
Continuous casting	2½ hours	48
Pressing	10 mins.	720

By inserting the consequences of x mistakes into the scale it is evident that high quality production management is essential. This need not necessarily be linked to the traditional selection of building tradesmen as foremen. The established supervisor tends to find the necessary disciplines as irksome. We have encountered factory managers ranging from ex-carpenters in their early twenties to M.I.C.E.s of 40 or so. This indicates the variable values placed by employers on trained expertise. We must, nevertheless, endeavour to cater for our clients according to their capabilities. The wide range within which our brief can fall, suggests a more flexible fee structure. We may merely advise on planning on one job whereas on others our involvement is complete. Having rationalised the structure, co-ordinated electrical and service requirements, prepared bending schedules and detailed ancillary hardware; having produced erection sequences and propping drawings, the establishment of a production schedule would seem to be a logical development of the service we could offer. Tradition has so far maintained that this is contractors' work and there is no accepted scale for payment nor awareness of its necessity.

Transport, handling and storage

How many engineers design precast units having given thought to the manner in which they are to be cast, struck, picked up, stored, transported, stored, erected and propped? Our contact with various manufacturers indicates that this is not normally considered to be part of the engineer's brief. One can draw the finished product in its final resting place and accept the tender price for getting it there. It is not uncommon that the most severe stresses undergone by the unit occur during one of these operations and that these stresses may be opposite to those encountered during its working life. The case histories of mistakes are numerous but typical of the type are Figs. 4 to 8. See right:

We should recognise that certain errors are bound to happen and introduce safeguards where necessary. Notes explaining our requirements are useful but, above all, we must first examine each component to see if there are any requirements.

The assembly of components

We find that, provided the manufacture and supply of components has been satisfactory, the assembly can usually be a straightforward procedure. The greatest difficulties are encountered in trying to match precast units to ostensibly accurate in situ work. It is here that one becomes aware of the considerable amount of adjustment that can be accommodated in an in situ job. Provided that no troubles are reported, engineers do not generally check on the dimensional accuracy of their buildings and, thus, tend to assume that they are obtaining the tolerances that they specify. Any good erector, be he working with steel or precast concrete, can carry out a certain amount of tailoring but there are strict limitations. There is nothing clever in asking for tight tolerances on mass-produced units and, in some instances, it can be dangerous. We always try to be as generous as possible.

Having established an erection technique, the principal problems involve the incorporation of other materials particularly in weatherproof jointing and insulation. There is a great temptation to resolve difficult precast problems by introducing limited sections of in situ work. This is rarely successful. The contractor cannot afford to carry the skilled labour necessary to produce a good job on such a small scale and the interference in the normal working sequence can be critical. A common problem arises when deciding what is to be done with the 50 or so units that have been incorrectly cast, each of which can cost £50. Often it is not until erection starts that the error is spotted. An acceptable but costly solution can usually be found, but a further requirement will then be to ensure that the factory knows of the mistakes. It has been known for an erection gang to be diligently 'correcting' a unit some 120 floors and two years after the original mistake was noticed. Feedback of information is vital.

There remains the particularly vexed problem of the degree of remedial work that is acceptable when the structural consequences are not serious. Our own view is that control exercised at the production end must obviate the greater proportion of making good. Contractors, according to their various philosophies on costs, may dispute this. We have no access to the figures of those who argue that one man can rectify faults in many units at less cost than may be entailed in slowing production to obtain quality. It often appears that teams of men are continuously at work correcting the faults of one man in the factory—or was he at the drawing board? Refinements in production techniques have been considerable and vast amounts of capital have been invested in plant by some contractors with the object of increasing productivity. The casual labour structure of the industry has resulted in a natural reluctance on the part of most employers to spend time and more money on training the necessary number of operatives. In consequence the responsibility of the designer to appreciate the practical problems is of paramount importance. Site management at all levels must now play an active role in dictating activities rather than merely guide a random number of gangs into a surplus number of operations.

A more detailed report of the pitfalls to be avoided would take several issues of the *Journal*, this resumé serves only to indicate that we do have troubles.

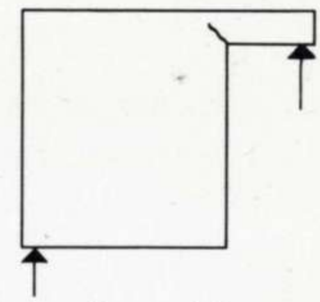


Fig. 4. A cantilever carefully supported in a frame.

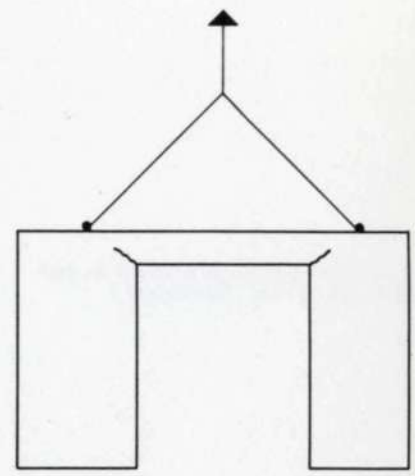


Fig. 5. A doorway producing a recognisable failure pattern in lifting.

Fig. 6. Slab supported in a stack parallel to its span.

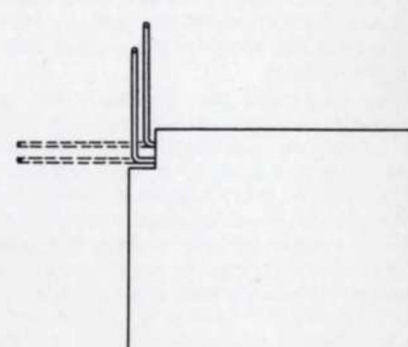


Fig. 7. High tensile continuity steel, bent up to 'avoid damage' in transporting.

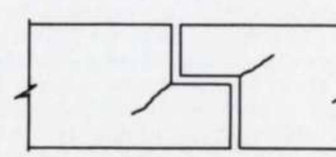


Fig. 8. A nib adequate in shear provided that it was not cracked in demoulding.

*D. Bishop—The Economics of Industrialized Building, B.R.S. Design Series No. 54

Area five development, Hulme, Manchester: from rationalisation to systemisation

S. S. Heighway

History

Our involvement in this scheme began in June 1966 when Lewis Womersley invited us to study his drawings, comment on the problem of general stability and give our views on the suitability of the scheme, on both technical and economic grounds, for complete industrialization.

The scheme comprised 918 dwellings, of both flat and maisonette types, located in four separate crescent-shaped blocks (Fig. 1). The flats, which were all 1 bedroom/2 person type, were confined to the straight sections adjacent to end conditions with lifts, which gave direct deck access at each level. Maisonettes occurred on both straight and curved sections, with deck access at lower floor levels only and they provided a wide range of dwelling sizes, from 2 bedroom/4 person up to 5 bedroom/8 persons. All maisonettes were of constant 17 ft. clear width so that variations in individual areas of dwellings could be achieved only by varying the positions of external walls at each level (Fig. 2). Further complexity resulted from the inclusion of more than one type of dwelling within any single 'stack' as defined by adjacent cross-walls.

It was obvious at the outset that whilst such planning principles resulted in a good spatial solution the structural complexities would undoubtedly mean abnormally high construction costs, regardless of the building method.

The architect and quantity surveyor both had relatively recent experience of a design at Basingstoke which was based on similar planning principles but was two storeys (i.e. two flats or one maisonette) lower in height. The principle applied to that scheme was design and tender by a short list of contractors on the basis of a performance specification and typical details, the assumption being that the majority of systems were only a way of producing concrete components in a rational way in a factory, that all had similar batteries and tables, and that individual design solutions would differ only in minor detail. The professional design team felt, therefore, that they could design a scheme which would be feasible for five or six contractors, which, with a measured bill would mean a straightforward traditional tendering process.

Ultimately the scheme was rejected by the Ministry because the lowest tender exceeded the cost yardstick, which was related to a fairly low density. The architect felt however that the cost differential alone would not have killed the job if there had been sufficient local enthusiasm for the basic principle of deck access.

As there was no resistance to this design principle in Manchester where greater density meant a higher yardstick and with experience of actual tender negotiations at Basingstoke, there was a general feeling of optimism for the future of the Hulme scheme up to the time when I was asked to report my opinions to a meeting of architects, quantity surveyors and representatives of the NBA, whose advice had also been sought.

Initial recommendations

From my preliminary study of the proposals I was quite convinced that no amount of ingenuity of design or construction methods could make this a viable scheme.

Certainly it could be built as detailed by the architect and principles of industrialization could be applied on a large scale, but the construction sequence would be completely disrupted by oversailing upper storeys, profiled ends to cross-walls, projecting local balconies and the too frequent occurrence of dissimilar dwellings on either side of a cross-wall, which created local external wall conditions with their attendant insulation problems.

The logical structural solution appeared to me to be a combination of in situ vertical structure and precast horizontal slabs, balustrades and wall cladding, but in my opinion the cost was likely to be outside the Ministry yardstick and considerable simplification of the layout was essential if we were to comply with the cost limitations.

When expressed at the meeting these opinions came as a disappointment to the project team who were at that time preoccupied with the question of tender documentation on the assumption that system building was a foregone conclusion. They had envisaged a situation similar to Basingstoke where architectural drawings were presented direct to a number of system builders and no consultant was involved. This approach to the problem presupposed that the scheme was very suitable for systemisation and that more than one system was applicable with only minor modification. This was certainly not the case at Hulme.

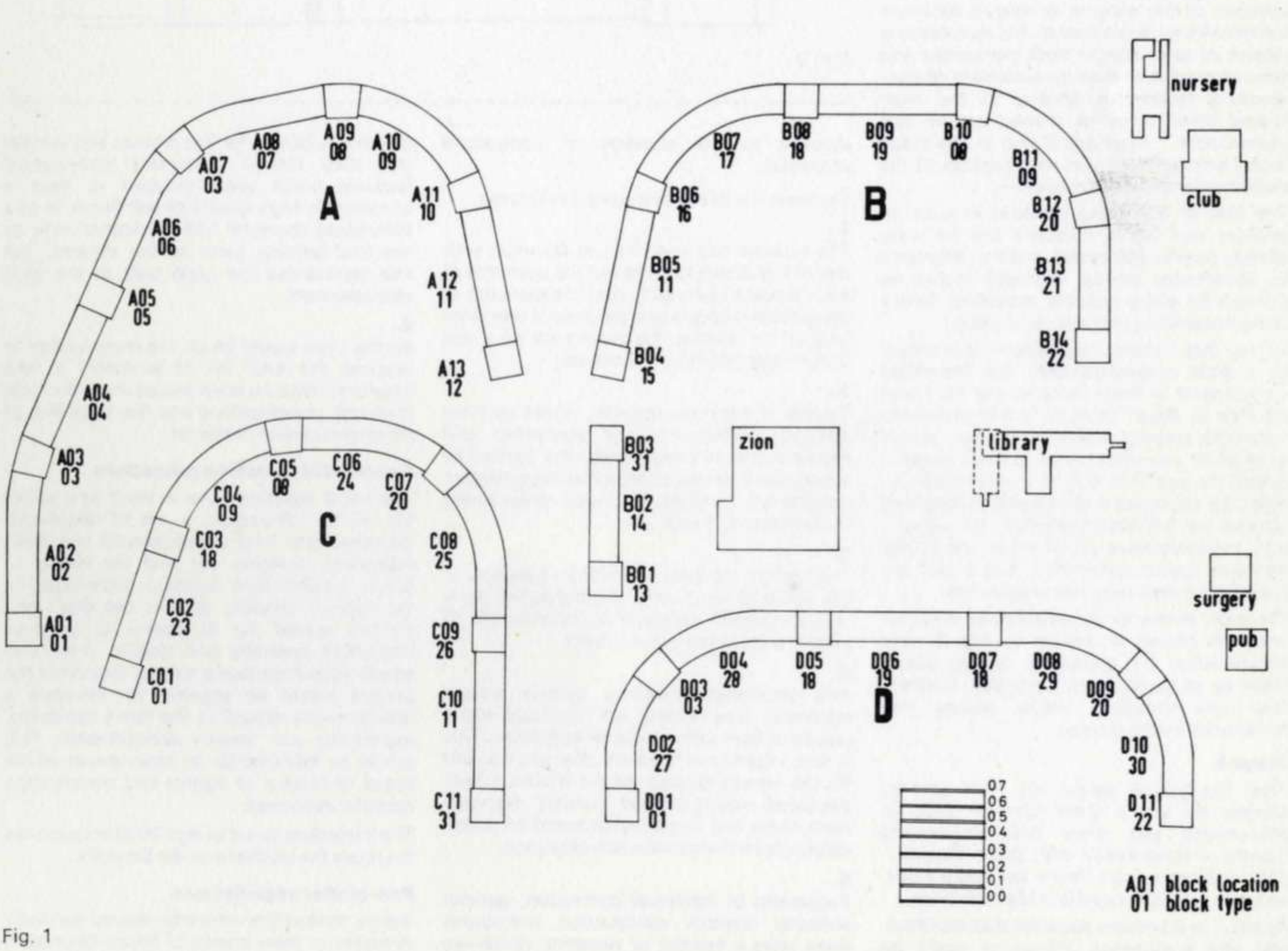


Fig. 1

Secondary stage

Up to this time our role in the development of the scheme was not defined—it seemed reasonable to assume that in the event of our being able to report favourable first impressions our activity might have been limited to the vetting of individual contractors' proposals in order to ensure that they were based on sound engineering principles both in design and construction techniques—it might even have ceased abruptly after acceptance of our report!

If our criticisms were accepted as valid a new situation was created—this indicated a major re-think, and consequently a serious interruption of the pre-tender programme. It was imperative that the scheme should be technically sound when it was offered for tender—it should be based on production and construction principles which would be attractive to more than one contractor and it should be sufficiently flexible to be capable of modification in detail to suit individual plant techniques and details.

Engineering problems, especially those involving, at one and the same time, design technicalities, simplification of construction, and minimal cost targets, can be solved only by very experienced engineers—in this particular case experienced not only in structural engineering in general, but having also detailed knowledge of the numerous specialist systems which might have application, to a larger or lesser degree, to the scheme under consideration.

Lewis Womersley paid us a very definite compliment in accepting without question our initial observations and inviting us to propose an immediate course of action which would minimise the programme delay and ensure that everything possible were done to produce an acceptable tender.

My own immediate comments were to the effect that the first step should be a rationalisation of the scheme to ensure minimum variation from a basic theme—hence maximum degree of repetition in both permanent and temporary works—then consideration of construction techniques leading to the most logical combination of precast and in situ construction. I must admit that at this stage in the proceedings I was very dubious of the viability of a fully precast solution.

The idea of our being involved in such an exercise was highly attractive but we were already heavily committed in other directions in Manchester so we naturally looked to London for every possible assistance before firmly committing ourselves to the task.

To say that London responded nobly would be a gross understatement! The immediate co-operation of Peter Dunican and Vic Kemp resulted in Bryan Seymour's being diverted from other pressing problems for what seemed to us an all-too-short period of three weeks—before the end of which, to my frank amazement, he produced a completely rationalised scheme for full industrialization, including a very comprehensive set of plans, elevations, sections, typical connection details and exploded diagrams of erection sequences.

The basic theme for the rationalised solution, which is shown in section in Fig. 3, was simplification by producing typical stacks made up of dwellings in a standard relationship and grouping similar stacks into continuous blocks on plan.

Stage 3

After this critical period we were able to allocate the job to Gerry Clarke's group in Manchester and there followed several months of close liaison with Bryan Seymour, with architects John Snow and Mike Hyde, and with services consultant Max Fordham.

As soon as it became apparent that architectural and engineering influences could be happily integrated we began to give serious

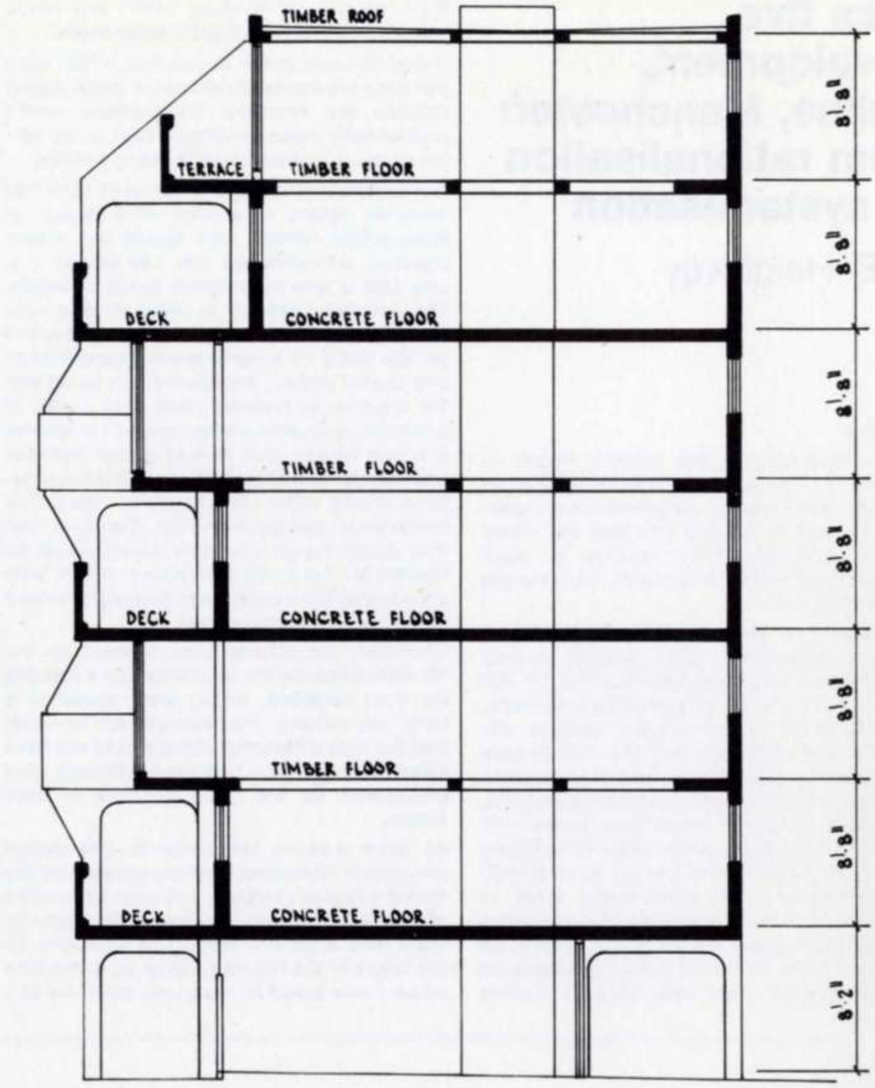


Fig. 2

thought to the question of contractual procedure.

Our main line of reasoning went as follows:

1.
The scheme had very little in common with any of the closed systems but the operators of such systems had a great deal of experience of the general problem and the project was large enough to warrant the setting up of a site factory specifically for the one job.

2.
Equally, if not more suitable, would be firms offering facilities for the precasting and erection of large components, and backed by a competent general construction organisation capable of handling the civil engineering content of the project.

3.
Contractors capable of handling all aspects of the entire scheme were preferable but there was no serious objection to the principle of precast supplies by sub-contract.

4.
Any proprietary building system whose economy was based on precision-made repetitive formwork would be applicable. This in fact, might have been the ideal solution but for the curved sections of the blocks, which produced wedge-shaped 'tunnels' between cross-walls and floors and imposed an added discipline on the construction sequence.

5.
Regardless of individual contractors' general attitudes towards construction techniques there were a number of elements which we would demand should be precast due to their obvious suitability for this process and the fact that they formed prominent architectural features which were required to have a consistently high quality of self finish. In situ techniques therefore had application only to the load-bearing parts of the scheme, but this represented the great bulk of the total concrete work.

6.
As the onus would be on the project team to propose the final list of tenderers it was imperative that the team should make the most thorough investigations into the suitability of firms for inclusion in this list.

Contractor selection procedure

The initial approach was to send to a select list of 13 contractors a set of architects' drawings and brief description of our basic intentions, pointing out that we hoped to apply industrialized building techniques to the highest possible degree, but that contractors would be at liberty to propose alternative methods and details. Any contractor who indicated a serious interest in the project would be required to complete a questionnaire related to the firm's capability, experience and known commitments. This would be followed by an interview at which broad principles of design and construction could be discussed.

The immediate result of this initial enquiry was to reduce the probable tender list to six.

Pre-tender negotiations

Before contractors were interviewed we made available to them copies of Bryan Seymour's drawings, presenting them as our logical interpretation of the problems involved and therefore offering a common basis for discussion.

Five of the contractors indicated a preference for a precast system. They all agreed that our approach appeared to be the most practical in the circumstances, and those having closed systems saw no obvious application for any particular system.

They were all aware that design team discussions would probably produce modifications but were happy with the assurance that the basic principles would be maintained.

The sixth contractor proposed in situ structure using the *Outinord* system but there had at that time been no practical application in the U.K.—before agreeing in principle we inspected jobs under construction in France and the initial stages of a Wates job in London, though this was not the contractor concerned.

There was, amongst all six contractors, a common attitude which served to establish our function in the development of this project. They were all prepared to enter into competition for the contract only if the variables were reduced to the absolute minimum—they strongly preferred not to commit themselves to the cost of pre-tender design and scheme development on an individual basis with only a 1:5 chance of winning the contract—particularly in view of the fact that they could see no logical alternative to our proposals.

The decision was then taken that we should be appointed as structural consultants with full responsibility for the structural design, all working drawings, specifications and supervision. During the pre-tender period all dimensional details, typical joint and panel support details, reinforcement and provision for built-in services would be determined by us and these would be the basis on which the bill of quantities would be prepared.

During this same period we would discuss with all contractors their individual proposals regarding precasting techniques, surface finishes to concrete, alternative materials and other points of detail. Within 14 days of receiving tender documents the contractors would confirm to the architect all such points which they had decided to incorporate in their tender, which, when submitted, would include all relevant cost adjustments. We, in turn would confirm our agreement with the individual modifications within a further period of 14 days and thereafter no further changes would be considered for purposes of the tender. In fact, individual discussions continued throughout the whole tender period.

It was quite obvious that individual discussion would bring to light some desirable basic changes which could apply to all contractors but each was assured that advantages which resulted from individual know-how and specialist method would not be passed on to the other competitors.

During the pre-tender discussion period two contractors backed out and were replaced by two others, neither of whom operated any system but had good precasting and general contracting experience. Both of these firms were extra to the original list of 13.

Tender documentation

It was agreed at an early stage that despite the inclusion of one operator of an in situ system one bill of quantities only would be produced. The superstructure would be measured on a component basis, supported by some component schedules, and key layout drawings.

In fact one bill was produced in which all the items were listed and generally described. This had to be read in conjunction with a supplementary bill which listed every prefabricated component, described each in detail and referred to the drawing reference for each component—these drawings also formed part of the tender documents.

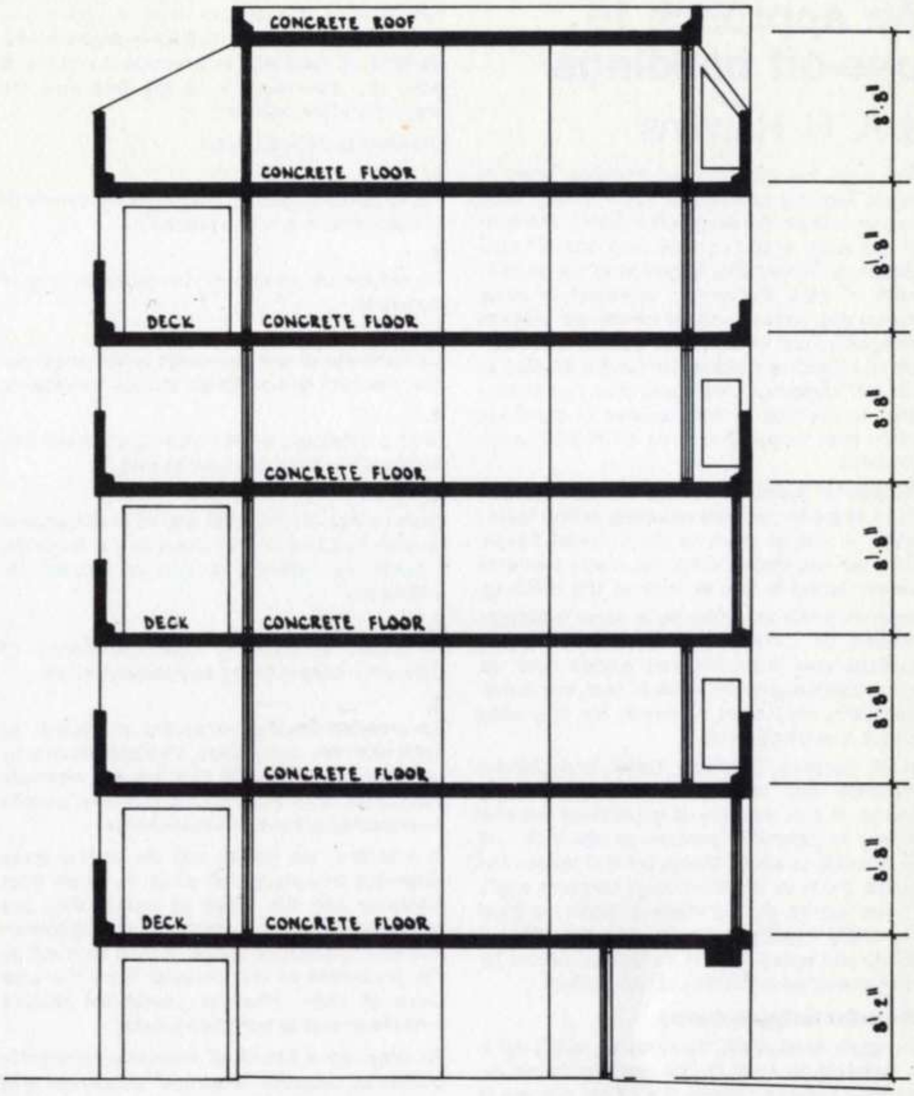


Fig 3

These bills were sent to all tenderers for pricing in exactly the same manner so that the *Outinord* operator found it necessary to employ a firm of quantity surveyors to effect the transfer of in situ measurement to precast billing.

Because of the close liaison during the pre-tender period and the very comprehensive nature of the documentation there were very few detailed queries from the precast operators during the tender period, but the in situ operator occupied a considerable amount of our time during this same period and, indeed, right from the time when he expressed a real interest in the project.

In terms of office production it was a relatively simple matter for us to produce special drawings for measurement on an in situ basis and we were quite prepared to do this if necessary but we succeeded in achieving a clear agreement on principles and details by discussion and correspondence. The main item which required special clarification was the reinforcement content, which, due to the structural continuity of the in situ scheme was different from the others.

Considerable pressures were exerted on us to eliminate shrinkage reinforcement which we included in the special steel estimates related only to the in situ work, the argument being that on the Continent the system always uses plain concrete walls.

Our insistence on the inclusion of this extra reinforcement was supported by the local authority but we were nevertheless relieved to find that the extra cost involved represented only about 1% of the amount by which this particular contractor failed to win the contract.

Conclusion

The procedure adopted for this project involved us in an abnormal degree of effort before invitation of tenders, the most influential factor being the inclusion of factory production drawings with the tender documents. This involved the issue of 9,000 A.3 size prints for structure alone for tender purposes and we could certainly not have achieved the target date without a carefully thought-out system of drawing production. Whilst this was based on the principles normally followed by the Housing Division there were a few minor modifications which Bryan Seymour agreed seemed particularly appropriate to the Hulme scheme.

The procedure has not resulted in any great increase in our total effort but has switched the emphasis from the later to the earlier stages of the work and in theory at least the post-tender period should be reasonably uncomplicated.

At the time of writing there is a major item of cost to be agreed which concerns the question of foundation design and the name of the successful contractor has therefore not yet been officially announced. Subject to agreement on the technicalities of the foundation problem the lowest tender is well within the Ministry's cost limits so that the very considerable effort put into the project by all members of the design team appears to have been well justified.

Architect: Wilson & Womersley

Services Consultant: Max Fordham

Quantity Surveyor: Cyril Sweet & Partners

An approach to one-off buildings

J. K. H. Hopkins

The case for systematized building scarcely needs arguing nowadays, but it is little used except in mass housing and schools. The bulk of our work is still in designing one-off type buildings. This article, largely from the point of view of total design as practised in Arup Associates, seeks to show where our skills as designers must be extended so that the benefits of repetitive production can be applied to one-off buildings. Arbitrarily one ought perhaps to say that it really applies to buildings other than housing and of £100,000 value upwards.

Traditional building methods might be defined as those methods requiring skilled handwork on site to produce the finished article. They are also messy, since the waste products are produced in situ as well as the building. Such methods are anomalous when buildings contain an increasing proportion of highly sophisticated manufactured goods such as air conditioning plant, control gear, windows, partitions, etc., most of which are degraded by dirt, damp and so on.

Most building contracts suffer from labour shortage and skilled men are particularly scarce. If the erection of a building became simply an assembly process on site it should be possible to speed things up and reduce the labour force to a semi-skilled erection team. These factors should make it easier to keep the team together thereby increasing their ability and speed. This is, in fact, borne out by the experience of Wates Ltd., and others.

Manufacturing industry

To apply production methods to buildings it is valuable to look at the manufacturing industry. In the first place, the whole process is usually under one control, if not actually under one roof. Secondly, actual production is largely by semi-skilled labour. All the real skill is concentrated in the design office, the tool room and in production management. (Toolmakers are the highly skilled and valuable people who are responsible for the machines, dies, etc., which are used by semi-skilled people to produce the bits which are put together on the assembly line by other semi-skilled people). Top management is in a sense the client because they decide what to make.

The building industry

In our industry the client is kept at arm's length from the rest of the business by the architect and consultants who are the designers. Production management is almost entirely in the hands of the main contractor and cannot be contacted until the design is virtually completed under the usual competitive tendering conditions.

Since the end product is a large immovable object assembly has to take place in situ which further sub-divides the contractors' production management.

The nearest equivalent to the toolroom are the mould-makers who operate off site and are often not available for dialogue with the designers.

Here are obstacles enough to the efficient production of anything, let alone a large complex mechanism such as a hospital or a laboratory block.

As designers we are the only bridge between the client and the production side and it is entirely our responsibility to see that he gets the best value obtainable in all respects. The client can only exercise his choice through us.

If we as designers cannot mitigate some of the drawbacks inherent in the building industry no one else can—unless they by-pass us.

If we take our responsibilities seriously we cannot allow this last possibility to take place, so we must look to our skills. We ought, by our designs, to facilitate and reduce the costs of both the manufacture of the bits and the assembly of the building.

We need to try and design:

1.
For large-scale use of the minimum number of components in any one building.

2.
To reduce the variety of fixing types and of materials.

3.
To eliminate all site processes which are slow, wet, messy or demanding of special conditions.

4.
With a thorough understanding of tolerances as they affect everyone concerned.

5.
With an eye for the selection of materials and finishes suitable to their place in the assembly process, i.e. relevant to first or second fix, and so on.

6.
To avoid, if possible, interdependence of different trades in fixing any particular item.

7.
To incorporate the necessary provisions to cope with site conditions. Component manufacturers, particularly if they are not normally associated with the building industry, cannot be expected to have this knowledge.

In addition, we could well do with a more extensive knowledge of other materials than concrete and the ways in which they are fabricated. Common methods of fabrication and their limitations matter at least as much as the properties of the material from the cost point of view. Physical conditions seldom limit the choice to a single material.

To draw up a list of all materials commonly produced, together with their properties and normal methods of fabrication is to realise that the scope is much wider than is usually exploited.

This country is particularly rich in small firms well practised in batch-producing in the sort of quantity, from tens to hundreds, as is likely to be needed in individual buildings. We only use a small proportion of those firms which are potentially useful and there are many making products only slightly dissimilar from some of the things needed in building. There is room here for a considerable broadening of the field and a lot of useful cross-fertilisation of ideas.

Lastly, and most important, the end product. There are already some notable results in this country from some of the approaches mentioned above. (1) They show that architecturally there is nothing to fear from simplifying the design and that a great deal can be gained on cost and time of erection. In some respects a levelling up of standards can occur. By concentrating on fewer components in larger quantities it is possible to afford better (2).

We cannot afford to go on detailing buildings with thousands of one-off situations. By the time the problem is apparent the architecture is already committed on the wrong tack and cannot be redeemed by a plethora of fine detailing. At best such details only make the building bearable to the users. Furthermore can we afford to build them?

References

(1) This week—'The Financial Times' industrial architecture award 1967. *The Architect & Building News,* 1967, *232* (23) 905-6

(2) School Construction Systems Development. *Architectural Design,* 1967, *37* (11) 495-506 *(The savings allowed the use of carpet throughout)*

The Medway building method

Werner Keis

Medway Buildings Ltd., Rochester, Kent, have been timber manufacturers for more than a quarter of a century and have over the years developed and built industrialized timber buildings ranging from schools, offices, hospital buildings, etc., to domestic housing. They have been our clients for many years and we have assisted them in the development of other Medway Building Systems.

However, although these buildings have been in great demand and Medway's building programme is in the order of £6m. to £7m. per annum, it was felt that the systems were too inflexible and imposed too many restrictions on architects who wanted to use them. The choice of cladding materials was exceedingly limited and the planning grid unnecessarily restricted, both factors resulting in dull monotonous elevations.

In order to resolve these problems Medway Buildings Ltd. decided to set up a design team. This team was inaugurated two and a half years ago and comprised CFD Partnership as design architects, ACP as consultant architects and Ove Arup & Partners as consulting engineers. Production specialists, quantity surveyors and specialists in contract management were supplied by Medway Buildings Ltd.'s own staff.

Brief

In order that the development team could work in close collaboration, it was most important that a general brief be agreed. At the beginning of 1965 the brief was decided upon, the main points being:

1.
To develop a building system suitable for 1 to 4 storey buildings covering virtually the full range of buildings except domestic buildings.

2.
To allow the maximum degree of freedom in dimensional choice.

3.
To allow the maximum freedom in choice of cladding materials.

In addition to these points it was essential that the system be economical in order to give it full sales potential. However, a system fulfilling all the points contained in this brief can hardly be called a building system as it is more a drastic rationalisation of existing building techniques to suit modern factory techniques.

General description of the method

The basic module adopted for the Medway Building Method, as it is now called, is 4 in., and provision has been made for the module to become 10 cm. when the building industry changes over to the metric system. The structural grid, both horizontal and vertical, is 1 ft. with a structural bay width of 6 ft. and/or 8 ft. for floors and 6 ft., 8 ft. and/or 12 ft. for roofs.

As Medway Buildings Ltd. are a timber manufacturing firm, timber has naturally been chosen for the structural frame, but due to structural limitation, and in some cases because of building regulations, timber has been substituted by steel.

The external walling is positioned on the face of the structural frame and the architects have the freedom to use almost any type of material ranging from timber or plywood to masonry or brickwork. Provided the 4 in. module is adhered to and the standard details which have been developed are maintained, the architect has an almost unlimited choice in respect of his elevations. The roof decks are factory-made plywood components with vapour barriers, insulation and bituminous felt.

In addition to the restriction of a maximum of four floors, there is a general height restriction of 50 ft.

Structural system

Within the Medway Building Method there are three distinct structural systems:

1.
A partition-braced timber beam and column system

2.
A frame system with steel columns and timber beams

3.
An 'incombustible' frame system with steel columns and beams with concrete floor units.

The partition-braced system will be used whenever possible, which in general will be for one and two storey buildings with a reasonable spacing of partitions. The partitions forming the bracing in the vertical plane are made in plywood panels. The horizontal bracing is formed by diagonals in the floor zones and by plywood roof panels. Neither the bracing partitions, nor any other partitions, are loadbearing, but to be effective bracing partitions are naturally restricted to column lines. The partitions are assembled in situ of the prefab element and are nailed and screwed to columns and beams, and bolted into the ground floor slab.

The frame system with steel columns and timber beams will primarily be used for large span single storey buildings, such as halls and gymnasia, three or four storey buildings, and two storey buildings which for planning reasons cannot be built in the braced partition system. In all these cases the horizontal forces are transmitted to the foundations by frame actions. This has led to the development of

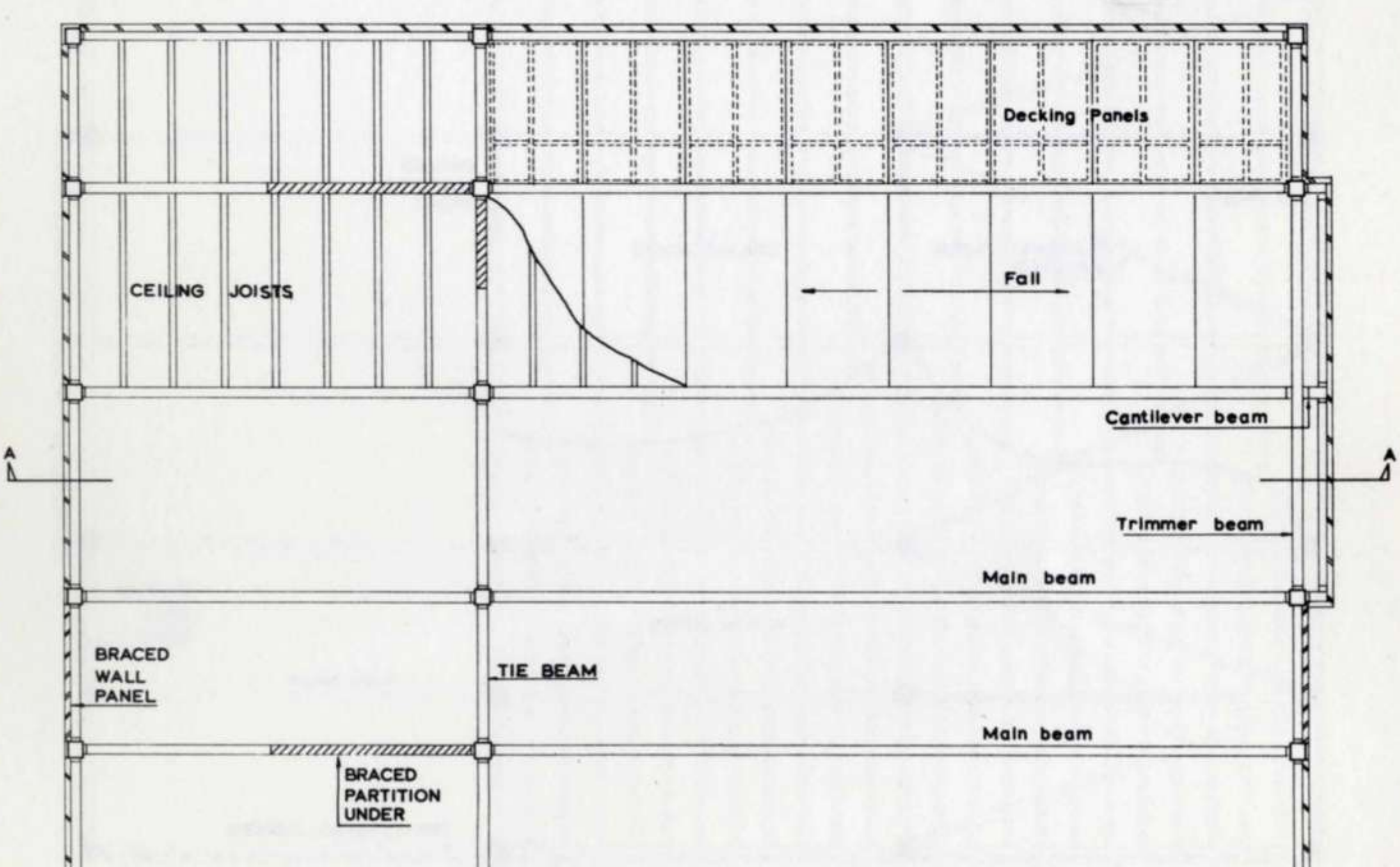


Fig. 1 Roof plan sectioned to show roof structure

the semi-rigid steel column/timber beam connection mentioned below.

The last of the three systems, the so-called *'incombustible'* system, will be used for institutional buildings within Purpose Group II of the *National Building Regulations* 1965 which calls for structures to be made of incombustible materials.

This system consists of the same type of steel columns as for the frame system, but is used in conjunction with steel lattice floor beams and *Siporex* floor slabs. The roof can be box beams and plywood roof panels, as for the frame system.

Timber columns are generally made of three pieces nailed/glued together and main beams and trimmer beams are plywood box beams. Solid floor joists span between the main floor box beams, and $\frac{1}{2}$ in. plywood on beams forms a floating floor. Prefabricated plywood components span the main roof box beams. Floor joists, floating floors and roof panels are all nailed in situ to the main structure.

The structure has been designed for floor loads of 60 and 100 lb. per square foot, and wind exposures up to D. All timber design has been carried out to B.S.112 and extensive use of the provisions in this Code has been made for full-scale laboratory tests on prototype structural members as an alternative or as a supplement to our calculations. These tests, which were all carried out by Forest Products Research Laboratory, fall into three major groups:

1.
Testing of various spans of standard box beams. As these box beams will be mass-produced it is important to achieve the most economical design which justifies the expenses for an extensive testing programme.

2.
Testing of braced partitions, for which no scientifically based design methods exist, and which have therefore been designed by approximate methods, which have been verified by laboratory testing.

3.
Testing of the steel column/timber box beam semi-rigid moment connection, which again can only be designed by approximate methods and verified by testing.

It is worth mentioning a few words about the moment connection referred to above. A great deal of thought has been given to the development of this connection, which, if in all steel or in concrete would be fairly simple to analyse and build, but a steel/timber connection becomes a serious problem. The problem is not so much to make it strong enough, but to make it stiff enough in order to keep the beam deflection within the permissible limits.

Publication of design information

The publication of all the design information to users has obviously been a major problem, which has been solved by publishing two sets of information: A *Design Guide* giving a brief description of the system has been issued generally to all potential users. This guide should contain sufficient information to enable the architect to draw up preliminary 1/16 in. scale sketch plans. For those who advance beyond this stage a more comprehensive *Design Manual* is available, which in three sections deals with design, components and site construction in more detail. This design manual should enable the architect to draw up a complete set of drawings.

For Medway's own internal use we have furthermore produced a *Structural Handbook*, which gives all structural information and diagrammatic sketches of the components.

Fabrication

All components for the buildings constructed to the Medway Building Method are either manufactured by Medway, or their supply is controlled and co-ordinated by Medway. The manufacturing of timber beams, columns, partitions, etc., presents no problems as these can easily be manufactured straight from standard drawings. This, however, does not apply to the extremely flexible external walling methods where positions of windows and doors, and choice of material will hardly make two wall panels similar. The detailing of such panels will therefore be time-consuming, and at an early stage of the development it was thought that in order to reduce the vast amount of routine work in this connection a computer program could have definite economic advantages. This computer program, which we are developing, is for Medway's use only. The input data for this program is co-ordinated to the elevations taken from an $\frac{1}{8}$ in. scale architect's drawing and the cladding materials. The output data consists of complete cutting lists and line drawings to be used for the manufacturing process.

The development of this program has not been an easy task. The original brief was simple, but additional requirements added during the period of development by the client made it very complex, but we anticipate the final program will yield worthwhile results for our client.

Conclusion

Two and a half years' development of the Building Method has come to an end and all connected with the scheme are now eager to see the results of their efforts when the Method is put into practical use. No doubt there will be teething troubles and changes will be required, but this is only to be expected with a scheme like this. Time will also tell as to whether clients will really make full use of the system's flexibility, or whether a system having much less flexibility would have been just as usable.

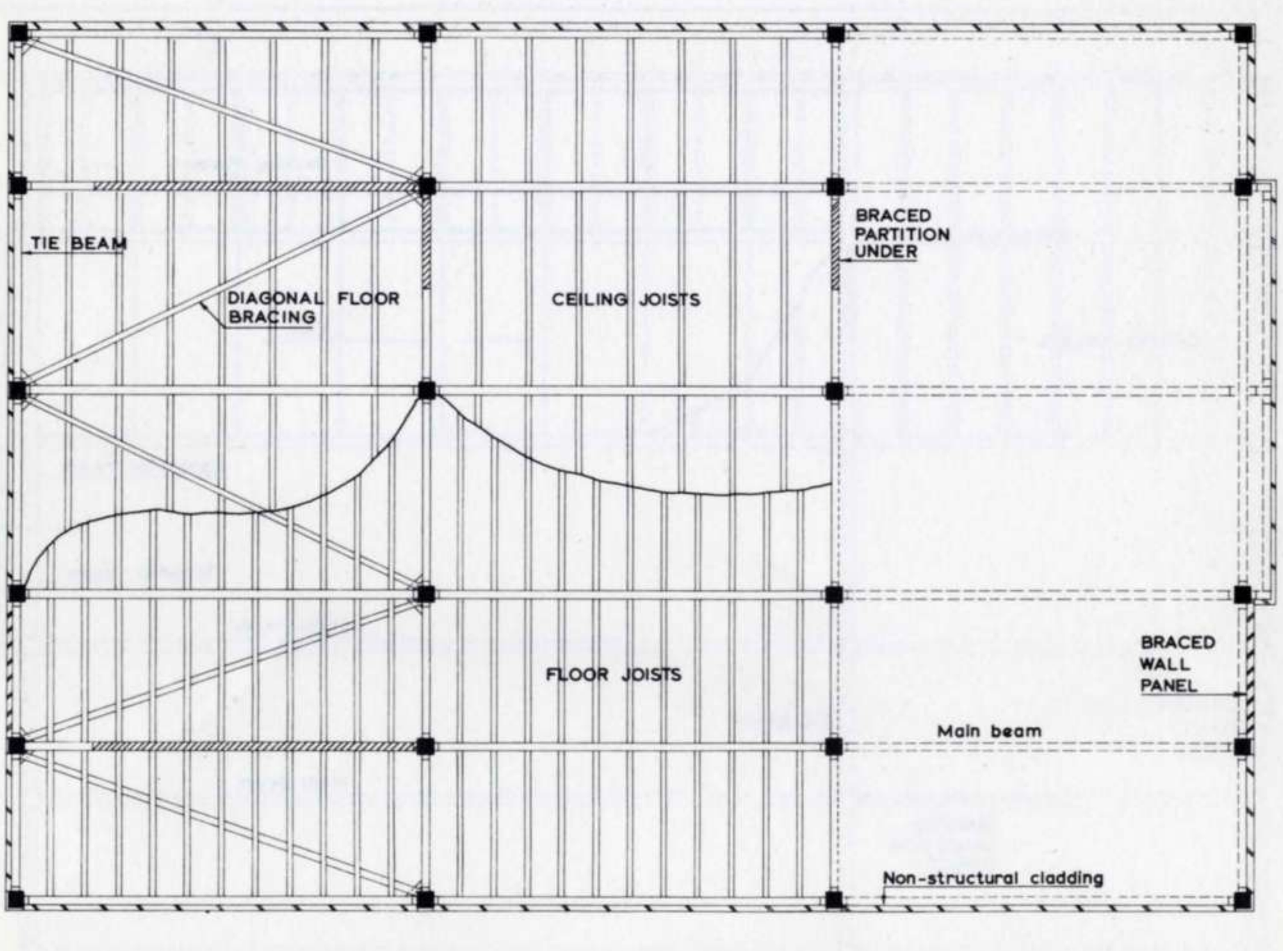


Fig. 2 First floor plan sectioned to show floor structure

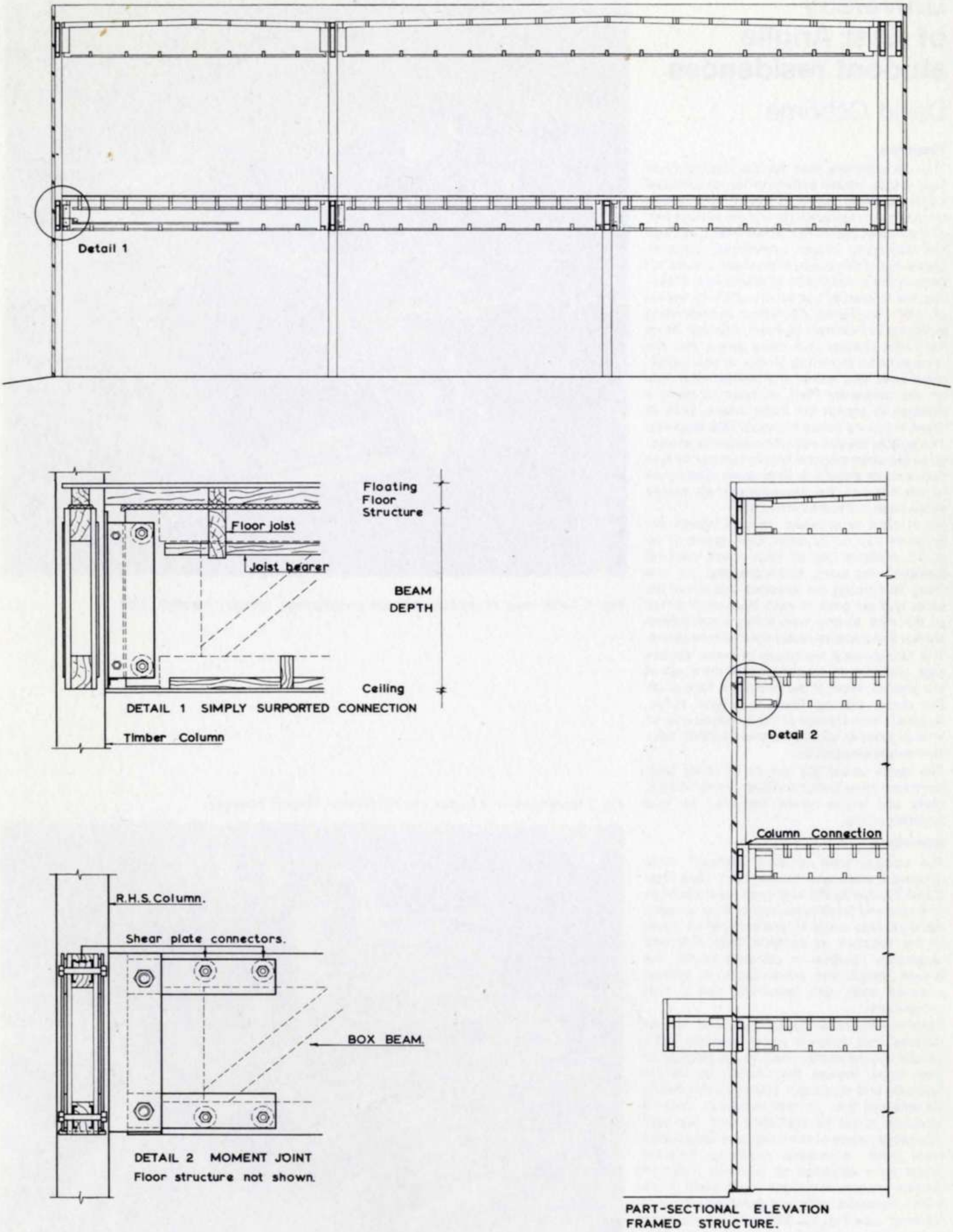


Fig. 3 Elevations and details

University of East Anglia student residences

David Osborne

Planning

The development plan for the University of East Anglia makes provision for an eventual 6,000 students, all of whom it is intended should be in residence on site for at least two of their three years. The Government, through the University Grants Committee, although approving of this policy in principle, would not finance the construction of residences. Therefore the University had to spend £1.75 million of their own funds, £¾ million in converting buildings at Horsham St. Faith, a former Royal Air Force Station five miles away, and the remainder on providing blocks of study-bedroom units and tutors' and married staff flats on the University Plain, in order to be in a position to accept the initial intake. Each of these locations caters for about 600 students. The bulk of the site accommodation is arranged in ten semi-ziggurat blocks laid out in two crescents on either side of reserved open space in the heart of the development, six blocks to the west and four to the east.

As at older universities, student rooms are grouped into social units. Each group of up to 12 students has its own snack cooking facilities, rest room, bathroom, etc., on one floor. The groups are arranged one above the other and set back at each level so that half of the roof at one level forms a convenient terrace (and access route) for the floor above. The blocks are a maximum of seven storeys high, reducing to six or five with the slope of the ground. Most of the bedrooms face south out across playing fields to a river valley. Access is from the rear of the blocks at ground level or from an elevated walkway at fifth floor level to a spine staircase.

The space under the terrace of study bedrooms contains bursar's offices, games rooms, plant and locker rooms and also car and scooter parking.

Structure

The cellular form of study-bedroom units dictated a cross-wall construction. (See Figs. 2 and 3). Due to the well-publicised shortage of bricks and bricklayers at that time, an early decision was made to prefabricate as much of the structure as possible. With 600 odd apparently identical or mirrored rooms, the greater speed and convenience of precast concrete units with everything cast in was recognised.

Thus the structure basically evolved. It was decided that external joints between units would not be mortar-filled in an attempt to hide them. Instead they would be treated honestly and expressed. Internal joints would be recessed ¼ in. so that individual units of structure could be identified and the prefabricated nature of the building distinguished from other monolithic buildings. External joints were designed to Building Research Station recommendations for drained joints with a neoprene baffle and damp-proof course backing (see Fig. 6). Structural connections were made with in situ concrete using ⅜ in. aggregate and were assumed to transfer shear forces. Where tensile forces were anticipated specially designed connections were used.

The thickness of the units varied from 10 in. external walls and 6 in. loadbearing internal walls to 4 in. non-loadbearing partitions and wardrobes. Slabs were 5 in. throughout.

For heat insulation purposes external walls were originally designed as a sandwich construction of concrete skins with polystyrene

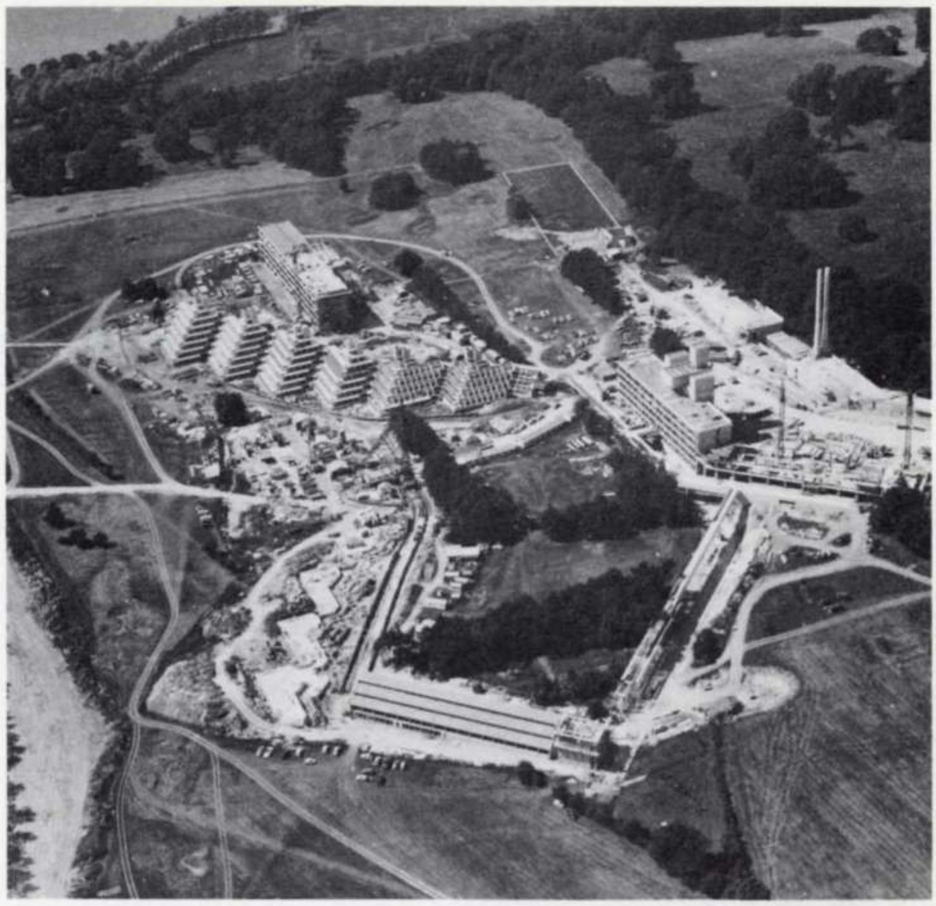
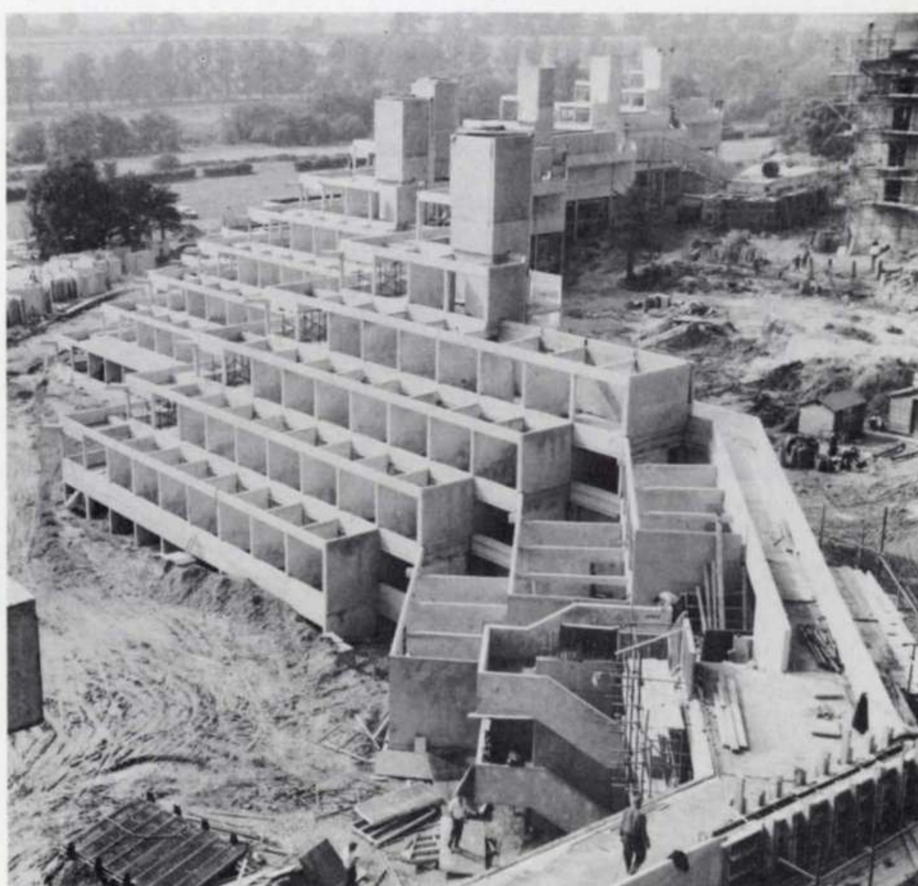


Fig. 1 Aerial view of residences under construction. (Photo: Aerofilms Ltd).

Fig. 2 General view of blocks 5 to 10. (Photo: Hugh F. Pocock)



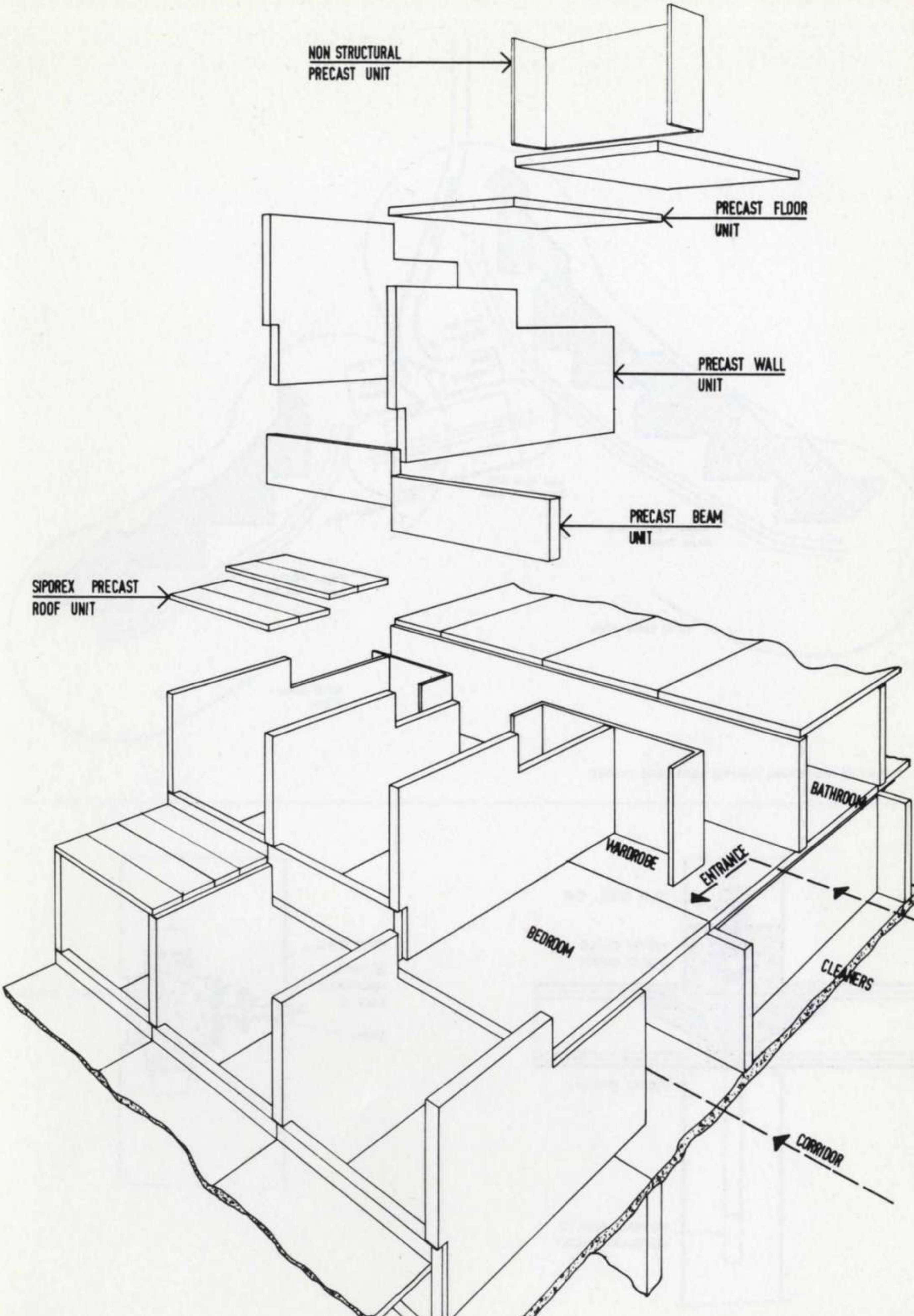


Fig. 3 Structural elements of residential blocks

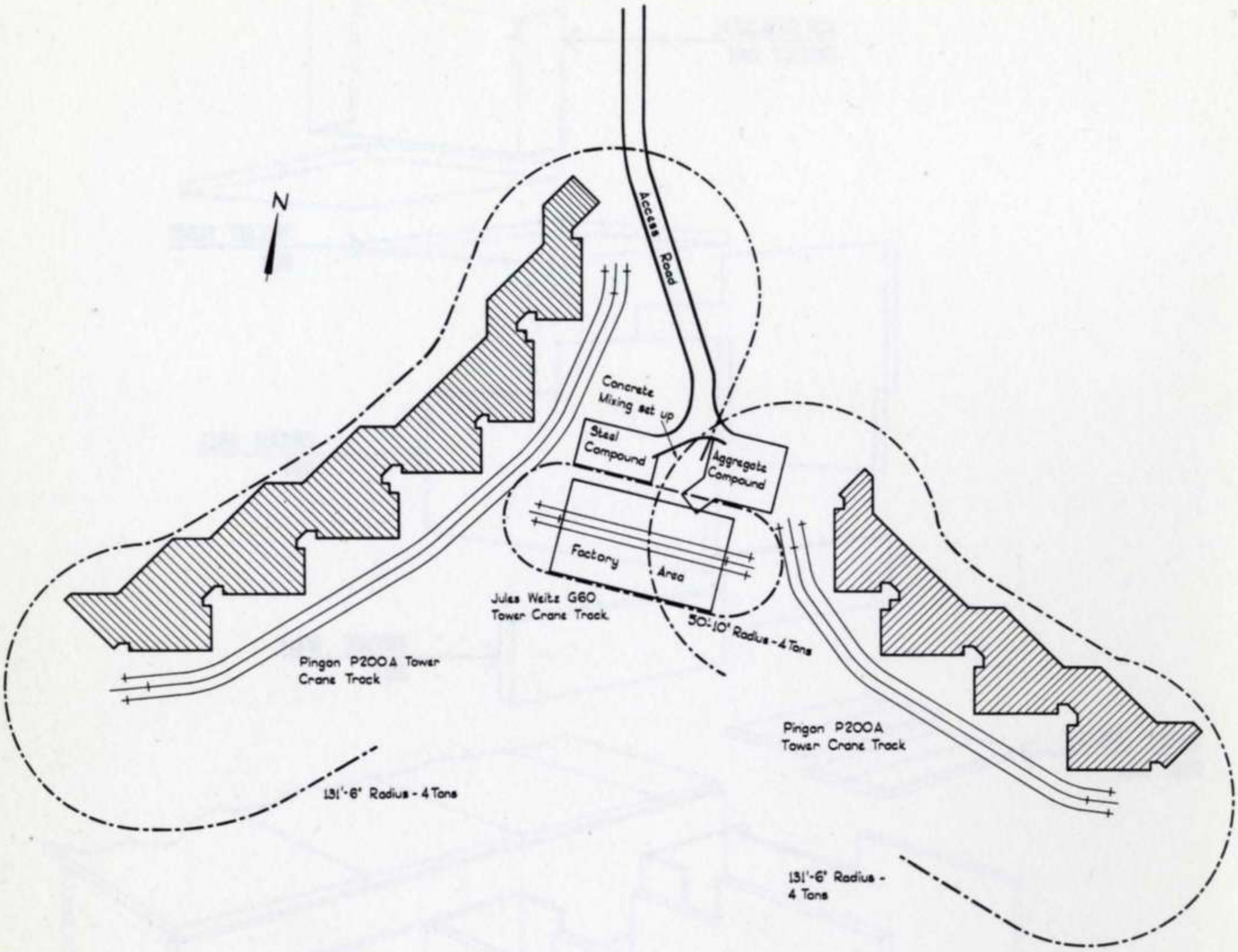


Fig. 4 Plan of residences, casting yards and cranes

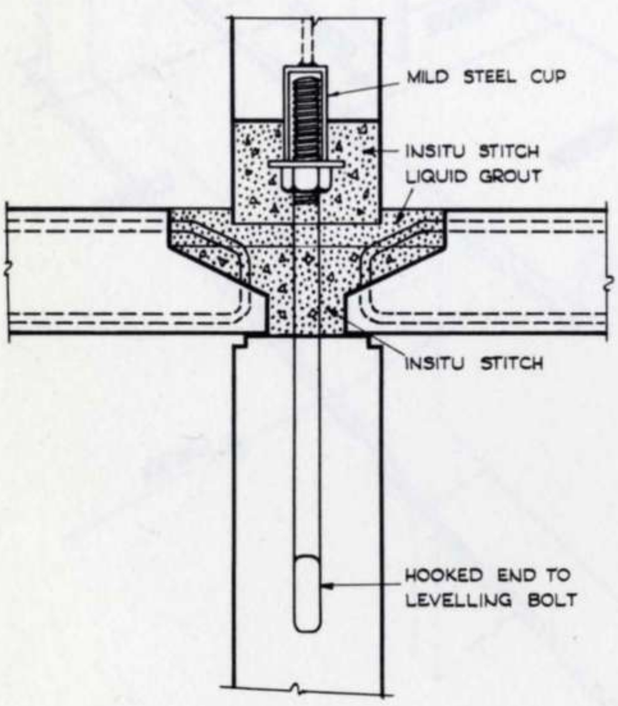


Fig. 5 Section through levelling pocket

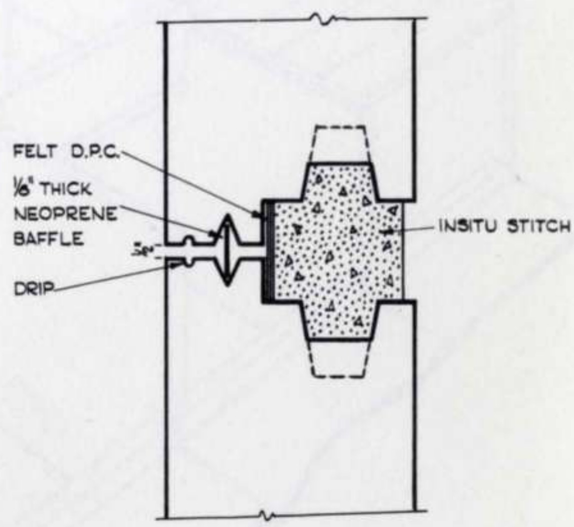


Fig. 6 Elevation showing vertical joint

filling but this specification was later changed at the contractors' request.

The architect decided to use plain concrete as a finished material, untreated externally but painted with emulsion paint on internal walls and ceilings. Floors were to be covered with fitted carpets or a plastic tile (no screeds).

Construction

When the basic form and structure had been settled we decided to invite a contractor to collaborate with us in the final design of constructional details, lifting hooks, jointing, etc. One was selected by competitive tender on a rudimentary bill and the design was detailed to his system with his help. At his request *Lytag* concrete units were substituted for the originally proposed sandwich construction external walls on the understanding that standards of finish, strength and thermal properties would be satisfactorily attained. Five tons at 120 ft. radius was quoted to be the limit of the crane's capacity and even though *Lytag* was used some of the units required 6 in. diameter voids cast in them to reduce the weight. Erection and plumbing up were simplified by lifting and levelling bolts cast into the tops of the wall units. (see Fig. 5).

Slabs were then lowered onto pre-levelled walls without any further adjustment.
An adjustable prop was designed to hold walls plumb until joints and slabs over had been completed.

Problems

All the units except for *Siporex* slabs specified for the terraces, were made on site in a 50 yd. x 30 yd. casting yard laid out as close to the residences as possible and erected by means of a *Pingon* P200A crane running on tracks from the casting yard to the furthest extents of the blocks (see Fig. 4). The majority of the wall units and most of the slabs were cast on edge in concrete batteries that were based on Building Research Station recommendations for the casting of units between concrete leaves (see Fig. 7.) Wall units which were angled on plan or excessively complicated had to be cast in steel moulds, while complicated slabs were cast in bed moulds with the upper surface float-finished. Because the tops of the typical cross-walls were complex, the contractor elected to attempt to cast them upside down. A steel tiliting table was made to invert them but it proved to be such a slow operation that he soon reverted to upright casting. To

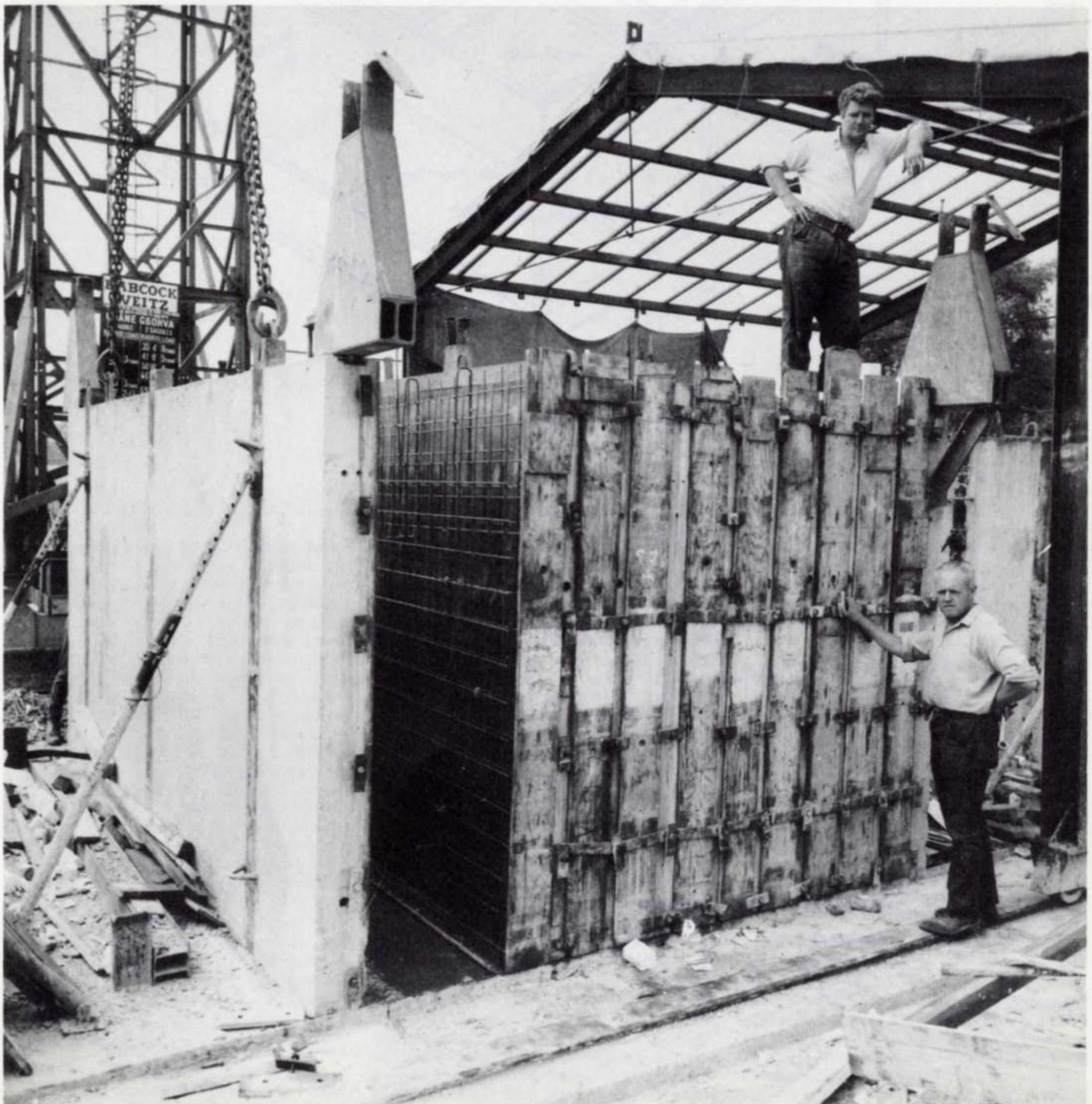


Fig. 7 Precasting battery
(Photo: Hugh F. Pocock)

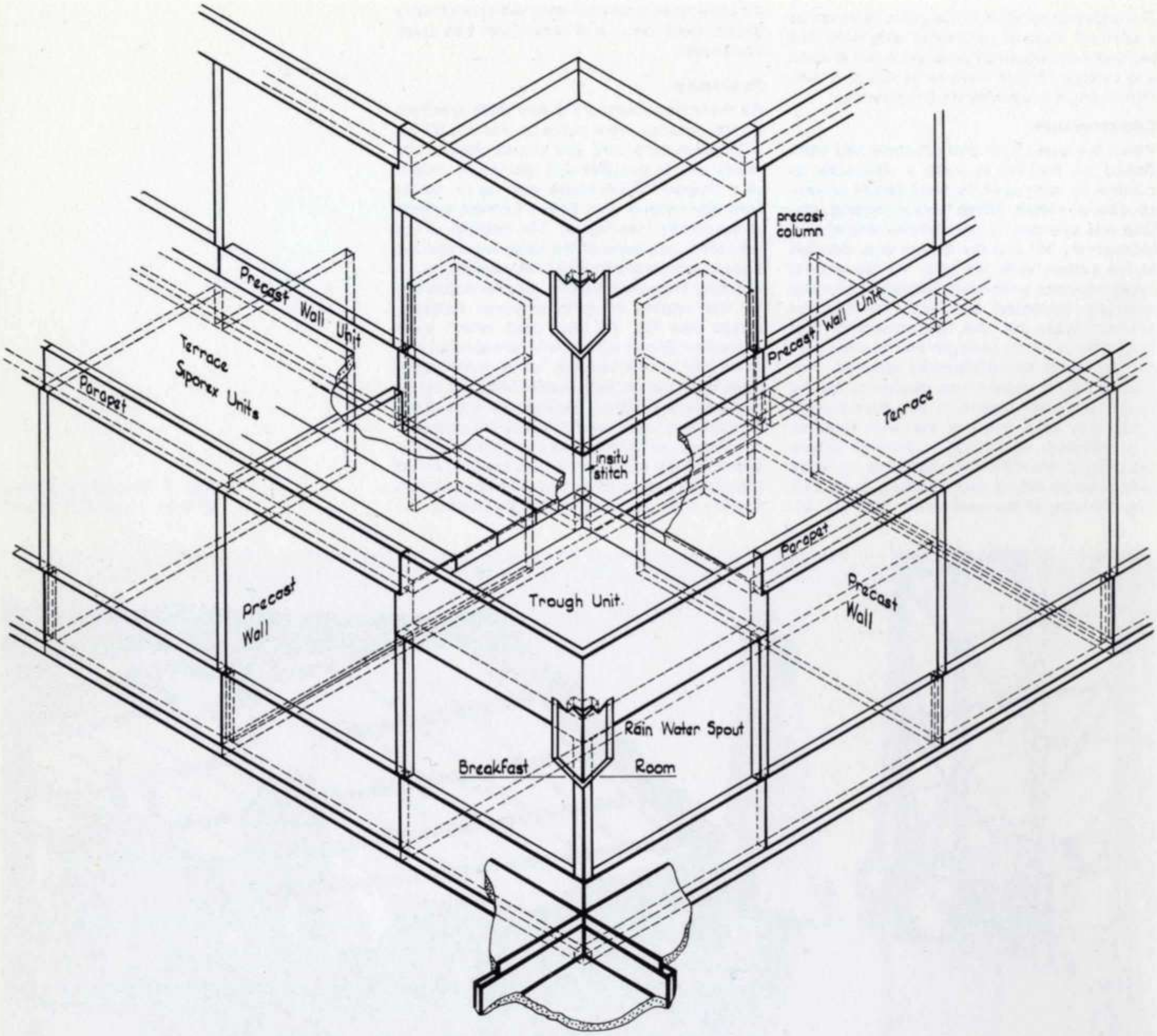


Fig. 8 Breakfast rooms—structural elements

overcome the difficulty of getting a good edge at the top of the wall units a rebated detail was agreed. It was not completely successful but did improve matters. Wall units were handled on edge from casting area to stock yard to building.

A lot of difficulty was experienced in achieving a satisfactory *Lytag* concrete which gave the required standard of finish combined with specified structural strength (3,000 lb/sq. in. at 28 days) and thermal properties. Also it proved to be fragile, large pieces chipping off at the slightest knock during handling and repairs are an eyesore.

Then, during a long spell of freezing weather, joints could not be concreted for fear of frost damage. The situation became critical when there were two levels supported on the levelling bolts only but a warm spell relieved our anxiety in enabling the joints to be filled.

In the light of experience, would we do it again?

There seems to be scope for precasting the repetitive bedroom units but far more of the structure which, as the design developed became non-standard, should have been in situ. It would not necessarily have been quicker to construct but certainly cheaper. Concrete batteries can produce an adequate finish economically provided that the units are simple and some care is given to the detailing and construction of edge details. No evidence of inadequate weatherproofing of joints has appeared apart from instances described later. Our greatest problem was to persuade services sub-contractors to accept the discipline of prefabricated construction and the subsequent tolerances. As experience becomes more widespread there is an increasing probability that these techniques can be used more effectively and economically, although to realise the full potential of such systems requires a radical reappraisal of the traditional design team relationship.

There appears in retrospect, to be a good case for getting it written into the precasting subcontract that the sub-contractor should be responsible for preparing the working drawings for the units, having himself acquired the necessary information from the appropriate sources as and when he needs it. This would tend to obviate a lot of the alleged delays and the misunderstandings that we experienced in our third party situation whereby we did the drawings and fed the information through as fast as we could make the revisions—and there were many.

We have had to investigate one complaint from the client which was partially a result of the predominantly southern aspect of windows with consequent high solar radiation effects which caused uncomfortablly high room temperatures. Coupled with bedding down of the building the resultant stresses caused movements in the building especially in the breakfast rooms where the structure is most complex (see Fig. 8). The movements manifested themselves, not surprisingly, in the mortar beds in internal joints and would probably have gone unnoticed had not the making good to the edges of units been done in one operation with the packing of the joints. As a result, in the structure taking up the stresses, the making good has pulled away very unevenly from the units leaving unsightly cracking. Also, faults appearing in the dressing of the terrace waterproofing, possibly due in some degree to a non-flexible membrane having been specified but certainly due to workmanship, allowed moisture to penetrate and appear through the cracks in a few cases.

At tender stage the cost of the precast structure was around £230,000. This figure was in fact exceeded but as a final account has not yet been rendered, it is not known by how much.

Architect: Denys Lasdun & Partners

Quantity surveyor: Davis, Belfield & Everest

Services consultant: Barlow, Leslie & Partners

Main contractor: Gazes Ltd.

Sub-contractor: Formcrete Ltd.

Fig. 9 Block 4 structurally complete (Photo : D. Osborne)

Fig. 10 The finished article (Photo : D. Osborne)

Brittle fracture of structural steel

Michael Griffiths

Introduction

The problem of brittle fracture of steel has been shrouded in a cloak of misunderstanding far too long. Brittle fractures have been recorded over the years in many types of steel structure; buildings, bridges, storage tanks, pressure vessels, etc. During World War II several brittle fracture casualties occurred in welded ships; more recently the King's Bridge in Melbourne, an all welded high yield steel bridge which collapsed in 1962, comes to mind, also the drilling rig 'Sea Gem' which collapsed in the North Sea in 1965 and resulted in the loss of 13 lives. Some brittle fracture casualties are shown in Fig. 1.

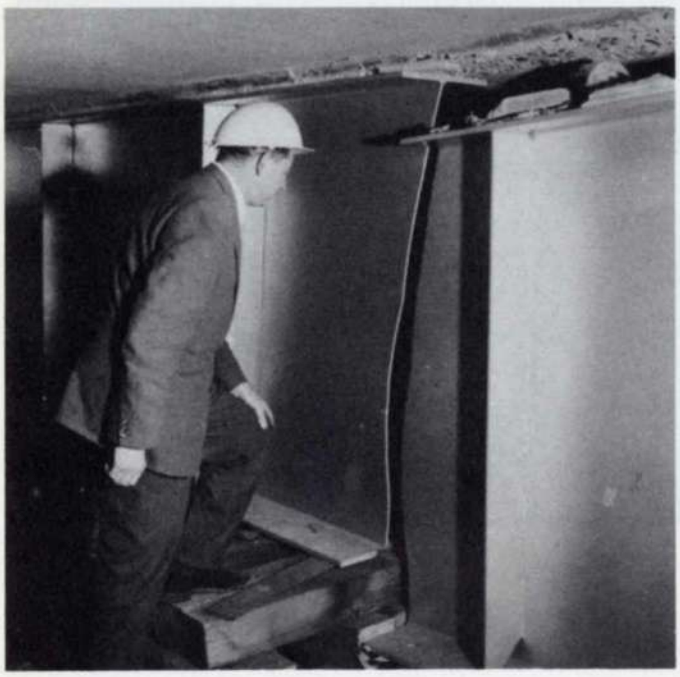
The risk of fracture can be judged from records kept over the last 20 years on two classes of structure, oil storage tanks and ships. Both show an incidence of 0.1 to 1 major fracture per year per million tons of steel used, with at least 10 minor fractures for every major one; for steel-framed buildings and bridges one can reasonably assume the risk to be of the same order. In addition Lloyd's data for postwar ships indicate some 18% more brittle fracture casualties for welded than for riveted ships.

Brittle fracture of a structure can be avoided by correct design just as failure by overloading can be avoided. Many engineers, however, are at a loss to know when brittle fracture should be considered as a design criterion or, if they do know this, then what to do about it. The purpose of this article is to attempt to clear up this situation, that is, to answer the questions:—what types of structure are prone to brittle fracture, and, how can brittle fracture of such structures be avoided?

The steels considered are limited to those used in normal constructional engineering, namely steels to B.S.15, B.S.2762 and BS. 968.

What is a brittle fracture?

A brittle fracture in steel has the characteristics of a crack in glass. It happens very fast and there is little or no plastic deformation surrounding the crack edges. It can occur at a stress level well below the yield value of the material. It is insidious because no evidence of brittleness is shown by the usual tensile and bend acceptance tests or by normal fabrication and working operations. In order to produce this kind of fracture in the laboratory the test piece must have a notch machined in it restricting the load-carrying cross section.



Figs. 1a and 1b King's Bridge, Melbourne. Welded girder bridge of B.S. 968 (1941) steel Collapsed 1962

Figs 1a to 1c Some brittle fracture casualties

Fig. 1c Petrol storage tank at Fawley of welded B.S.15 steel Collapsed during pressure tests, 1952

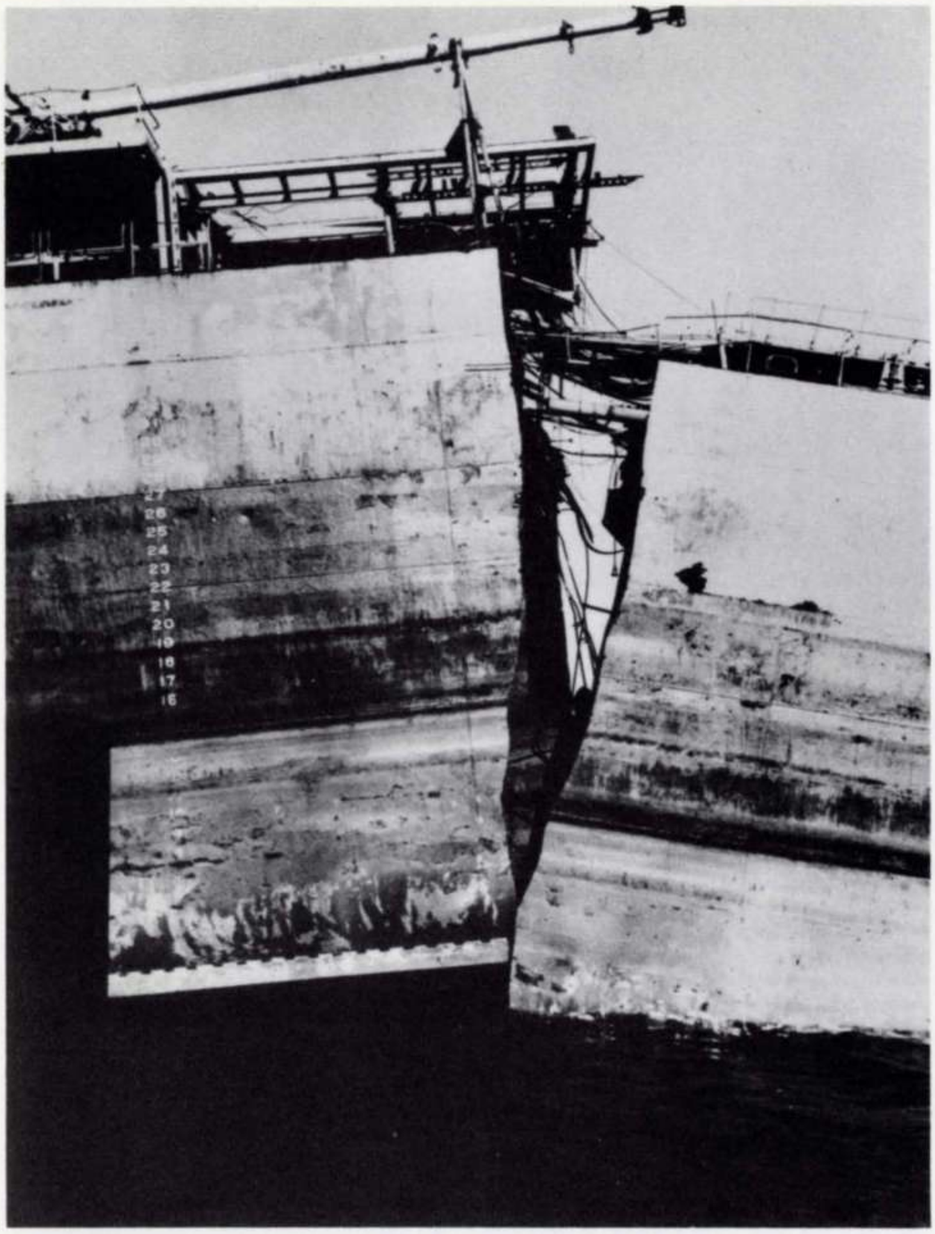
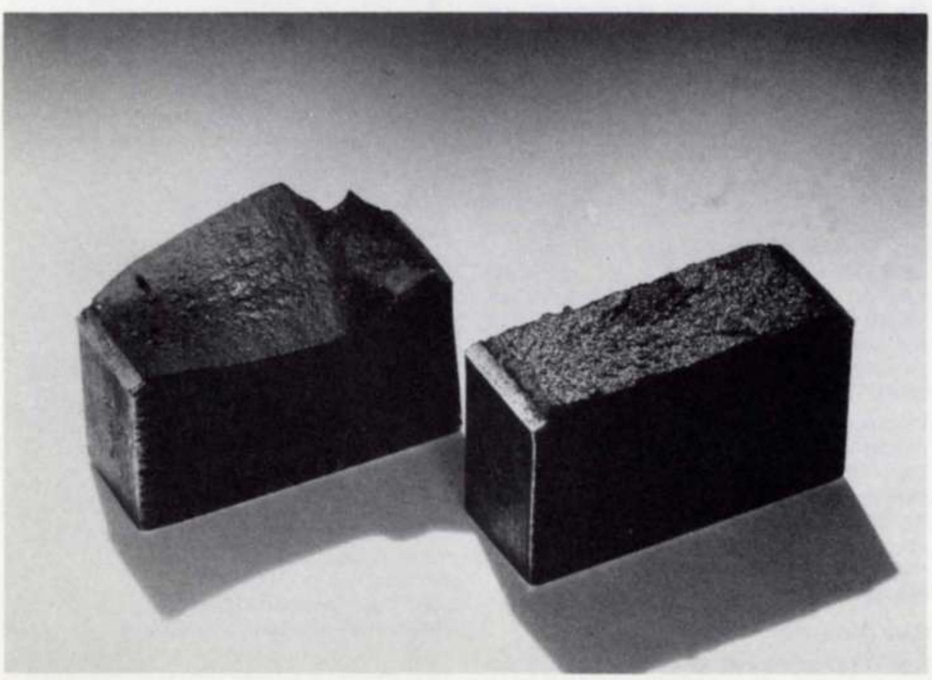


Fig. 1d Welded European Tanker. Fractured in calm seas, 1942

Fig. 2 Ductile and brittle fractures of notched tensile specimens (2)



Brittle fractures are confined to materials subject to a tensile load which can fail along both the cleavage planes and the slip planes of the crystal structure. A brittle failure occurs along the cleavage planes and a ductile failure along the slip planes. The propensity of a steel to cleavage failure is associated with a lowering of temperature. A metal finds it easier to fail along the cleavage planes at low temperatures.

Fig. 2 shows a pair of notched tensile test pieces of the same material. The specimen on the left has failed in a ductile manner along the slip planes. The fracture surface is inclined to the direction of load and is dull and fibrous, the crystal planes have slid over each other thus showing nothing of the crystal structure of the material. The specimen on the right has failed in a brittle manner along the cleavage planes. The fracture surface is bright and crystalline and at right angles to the direction of load. The specimen on the left was tested at 50°C while the other was at 44 C. No structural change is involved in the two cases. The difference in behaviour is quite reversible, a specimen which was brittle when cold will become ductile when warm. In the past the appearance of the cleavage fracture surface might have been interpreted as worn out steel which had become crystallised. This would be completely erroneous. It is a fact of life that steel which is usually regarded as an extremely ductile material can become brittle at low temperatures. Having established this we must live with it and know what to do about it.

It is obviously important to know the temperature below which a steel will fail along its cleavage planes and above which it will fail along its slip planes. Actually there is no one temperature but a temperature range in which this transition takes place. This temperature range is known as the critical temperature range, or more usually, the transition temperature range. There are a variety of tests available for determining the transition temperature range of a material. They do not, in general, give the same results but this is not surprising when the facts affecting transitional behaviour are considered, as we shall see later. Only the Charpy V-notch test is described here as it is the most universally used.

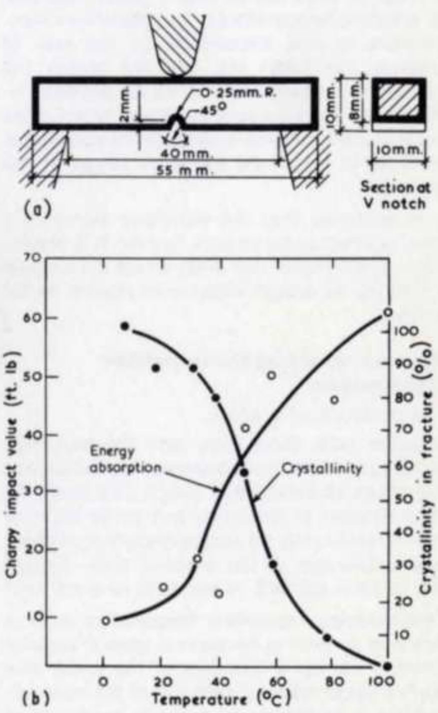


Fig. 3 Charpy V-Notch specimen and test results
(Illustrator: Margaret Woodward)

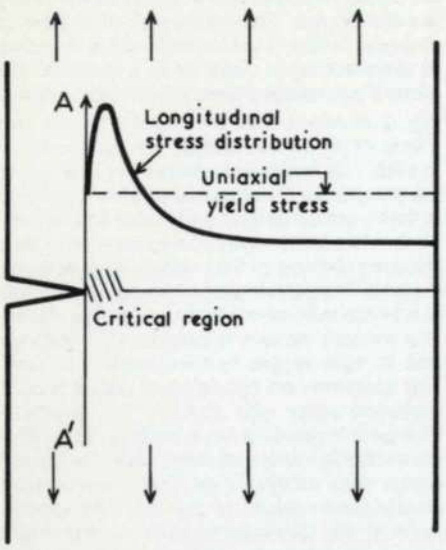


Fig. 4 Stress distribution
in a plate at a notch
Illustrator: Margaret Woodward

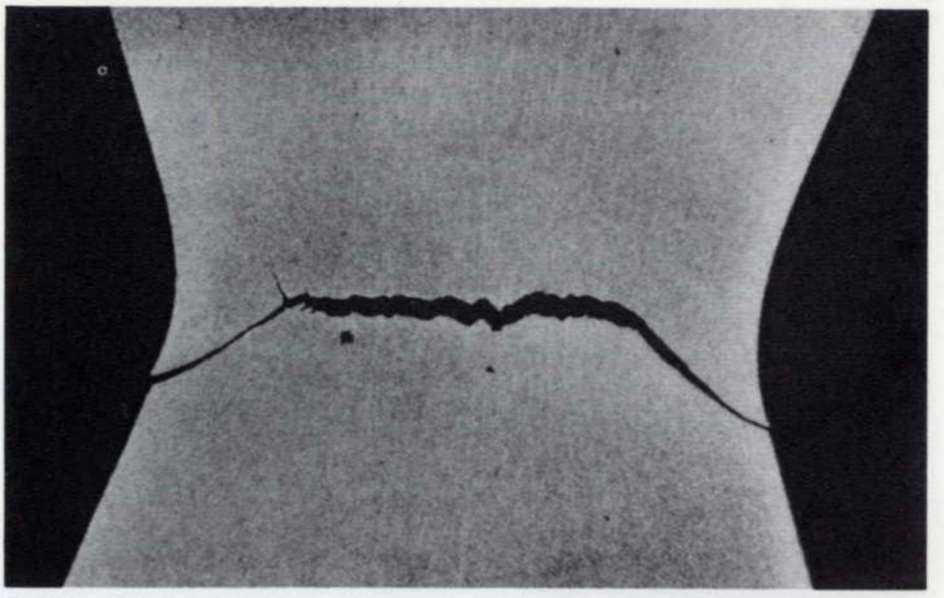


Fig. 5 Cup and cone fracture
of round tensile specimen at room temperature(1)

The Charpy V-notch test

This is an impact test. A series of small notched bars, (Fig. 3a) is broken with a pendulum blow; the bars are supported at their ends, struck at the centre and tested at different temperatures. The conditions for carrying out this test are described in B.S.131: Part 2: 1959. The energy absorbed by each bar is measured from the difference in pendulum swing before and after impact. Also the fracture appearance is noted in terms of the percentage of crystalline or shiny material in the fracture surface.

A typical plot from a series of such tests is shown in Fig. 3b. The energy absorption decreases rapidly from approximately 60°C to 40°C and the percentage crystallinity increases rapidly in this temperature range indicating a progressively brittle mode of failure.

This test uses a notched specimen and impact loading. As already mentioned the presence of a notch is essential to obtain brittle fractures at ambient temperatures. The transition temperature is also dependent on the rate of loading, the faster the rate the higher the transition temperature. Hence a notched impact test is a convenient quick test which does not involve excessive cooling of the specimens in order to cover the transition range of the material.

It is apparent that the transition curve of a steel is affected by various factors. It is essential to understand how these affect transitional behaviour to design effectively against brittle fracture.

Factors affecting the transition temperature

The presence of a notch

A notch does more than raise the transition temperature of a steel. Its presence, because of the stress concentration effect, increases the local stresses at its root so that the stress level in this region may be approaching the ultimate tensile strength of the material even though the general applied stress level is quite low.

The increase in transition temperature when a notch is present is because a triaxial state of stress is set up at the root of the notch and such a stress state inhibits slip of the material. A plate in tension with a notch machined in it is shown schematically in Fig. 4. The stress distribution across the plate at the notch is shown. High local stresses are present near the root and these give rise to contractions in the other two directions, across the plate width and along the length of the notch. These contractions are resisted by the volume of material to the left of the line AA, which is unstressed and hence lateral tensile stresses are set up.

That a triaxial stress system inhibits slip can be shown by a normal tensile test carried out at room temperature. In Fig. 5 the fracture region of a round tensile specimen is shown after failure. Gross local yielding has taken place causing the familiar 'necking', but the start of fracture is at the centre. Because of the 'necking' the longitudinal stresses are inclined to the axis of the specimen and so have radial components. These radial stresses inhibit further slip and the fracture is along the cleavage planes.

Hence the presence of a notch results in a raised transition temperature and high local stresses. If the material is ductile in the notch region then local yielding will relieve these high stresses. But if the material is below its transition temperature the elastic stresses may be extremely high and the critical region shown shaded in Fig. 4 a starting point for a brittle crack.

Thickness of material

For a given steel the transition range will vary with material thickness. There are two effects of thickness, a metallurgical one and a geometric one. The metallurgical effect is due to the differences in rolling temperatures and cooling cycles for thick and thin material. The size of the individual crystals (grain size) is smaller in thin than in thick material because of the greater rolling deformation and temperature cycling during rolling. The smaller the grain size the better the mechanical properties in general and in particular, the transition range for thin materials is lower than for thick. The effect of grain size on transition temperature is illustrated in Fig. 6a which shows a plot of temperature for 20 ft. lb. energy absorption for Charpy keyhole specimens—these are similar to the V-notch specimens but have a keyhole-shaped notch machined in them—against grain size for two ship steels.

The second effect is purely geometric in which thicker material tends to have a higher transition temperature than thinner. This is demonstrated in Fig. 6b which shows the results of a series of notched bend tests. The specimens tested were machined from the same initial plate of low alloy steel and hence the results represent the geometric effect only.

Rate of loading

It is well known that specimens subject to a sudden application of load show a marked increase in yield strength over that of the normal 'static' loaded specimens. This increase in strength is accompanied by a loss of ductility. The material becomes less 'tough' and more brittle under dynamic loading. Hence high strain rates raise the transition curve of a steel. This is borne out in practice. A brittle crack is more difficult to start (i.e. static loading) than to propagate (i.e. dynamic loading) and a brittle crack once started will pass through material which would be above its transition range for static loading conditions.

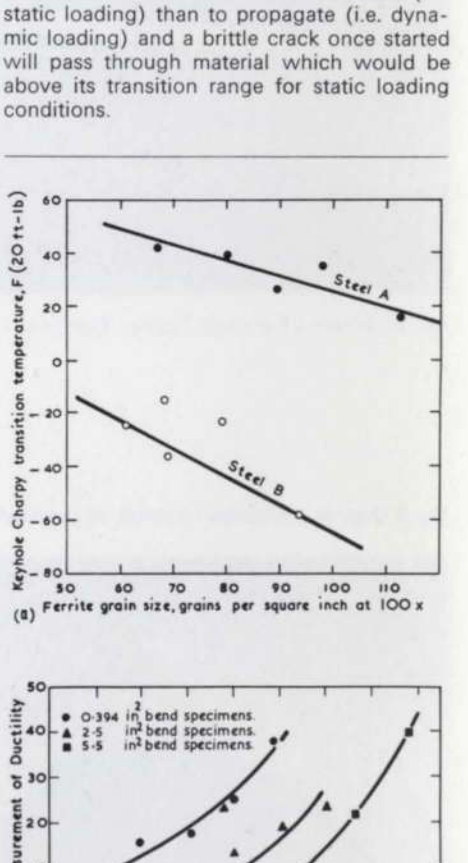


Fig. 6 Effects of material thickness
on transition curves
(a) Effect on grain size(4)
(b) Purely geometric effect(3)
(Illustrator: Margaret Woodward)

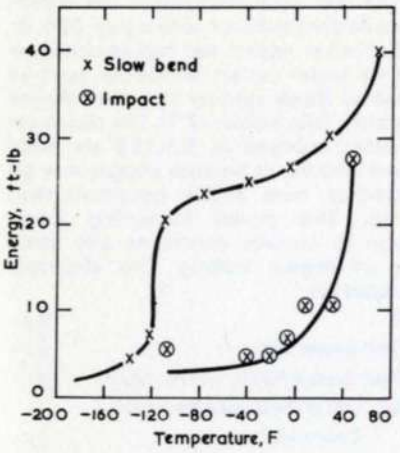


Fig. 7 Effect of rate of loading on transition curves(4)
(Illustrator: Margaret Woodward)

The influence of the rate of loading on the transition temperature is shown in Fig. 7. Both the slow bend and impact tests used standard Charpy V-notch specimens. The results shown which are for mild steel are fairly typical of many steels. The impact curve is seen to be the more critical. The points for the slow bend test were obtained from the load-deflection curves.

Why welded structures?

All that has been said applies to steel structures in general whether they are fabricated by bolting, welding or riveting. Brittleness in steel need not be a terrible thing provided it is not accompanied by any loss of strength. A steel which fails below its transition range will always fail along the cleavage planes, but if this is accompanied by prior yielding and a stress at failure above the yield stress of the material, then there is no cause for alarm. If, on the other hand, there is a loss of strength so that failure occurs at a low stress level well below the yield value of the material, then there is cause for alarm. For practical purposes brittle fractures of this type are confined to welded structures although they can occur in bolted or riveted structures. What is so special about welded structures? Partly it is their monolithic nature. Brittle cracks may be stopped by a bolt hole or by the end of a plate whereas these discontinuities may not be present in a welded structure. Mainly, however it is the metallurgical upheaval which takes place at the welds in both the weld material and the adjacent plate material due to the rapid cooling of the molten weld metal and the restraint provided by the plate which results in welded structures being susceptible to low stress brittle fracture.

The restraining effect of the plate induces tension stresses in the weld region both along the weld and laterally. The thicker the plate the greater the restraint and the higher the induced stresses. Also in thick plate, stresses will be developed in the direction of the thickness of the plate. These built-in or residual stresses may be as high as the yield value of the material. They are balanced by compressive stresses in the plate away from the weld. Typical residual stress distributions for a plate with a central butt weld are shown in Fig. 8. These stresses can be considered in the same way as stresses due to applied loads in that they are not localised but spread over a large volume of material, and are directly additive to any applied stresses. Hence at a low applied stress the actual stress in the weld region may be high.

The microstructures of the weld metal and the metal at the fusion boundary between the weld and the parent plate (the heat affected zone) are a result of the rapid cooling and plastic straining of the molten weld metal and this material may have transition curves at substantially higher temperatures than that of the plate material away from the weld. It is found that different parts of the weld region have varying toughnesses. In addition this region will undoubtedly contain flaws in the form of microscopic cracks which are in effect extremely sharp notches.

Summarising, the weld region is one in which the material contains sharp notches and has elevated transition ranges compared with the plate material, and is surrounded by a high tensile stress field. All these factors contribute to make the weld region sensitive to the formation of brittle cracks. Such is the case in practice. Cracks initiate in the welds and propagate into the plate.

Heat Treatments

It is possible to apply some ameliorating heat treatment to welded fabrications. The most obvious one is thermal *stress relieving*. This is carried out at between 580°C–650°C. At these temperatures the yield strength of low carbon steels is of the order of 1-2 tons/in^2 and there is a significant creep rate. If the whole of the assembly is brought to this temperature range and kept there for roughly one hour per inch of thickness of material, residual stresses are relaxed everywhere. If the assembly is then cooled uniformly no further stresses are induced. Stress relieving also removes some forms of local metallurgical damage and lowers, by as much as 50°C in some cases, the transition ranges in the weld region. It is interesting to note that no low stress brittle fractures have been produced on stress relieved welded specimens in the laboratory for the structural steels considered here. It is worthwhile recording however that

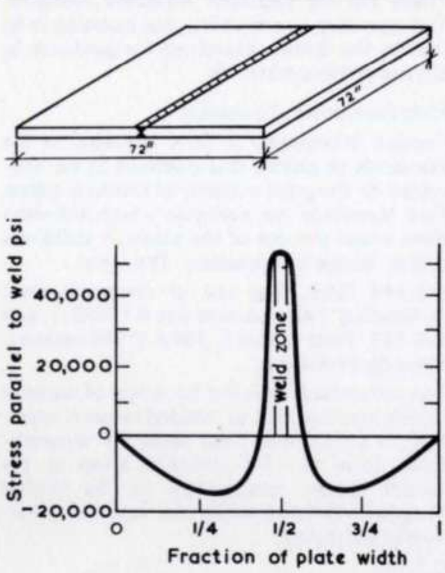


Fig. 8 Residual stresses in a plate with a central butt weld(4)
(Illustrator: Margaret Woodward)

stress relief heat treatments of alloy steels can produce deleterious effects unless the heat treatment conditions are carefully chosen to avoid this. Also the benefits of stress relief apply only to fracture initiation (this is discussed later) and that the lower transition temperatures shown later in Table 3 do not apply to propagation. It is often impossible to stress relieve whole assemblies, but local thermal stress relief may be carried out by flame heating, blankets, etc., but this has only limited application and can result in increasing the residual stresses.

Normalising of low carbon steels is carried out by heating to about 900°C followed by cooling in air, and causes re-crystallisation. The grain structure is refined and as has already been mentioned, this has the effect of lowering the transition temperature. It is very rarely possible to normalise the whole structure or part but normalised plate can be used. If normalised plate is used in a structure, although the material adjacent to the weld has been further heat-treated by the welding, it has a lower transition range than the same material which had not been normalised.

Significance of the Charpy Test

After what has been said, the question arises as to the relevance of tests such as the Charpy to the behaviour of real structures. The answer to this is that there is, in general, no correlation between the transition curves obtained from the Charpy test and those obtained from large scale tests which simulate service conditions. On further consideration one would expect this to be the case—the Charpy specimen is small and so contains little or no residual stress, is usually taken from the parent plate and is subjected to an impact load—conditions quite unrelated to those at the critical regions in the welded joints of a large fabrication subjected to static loading. Sometimes however a correlation does exist between Charpy results and service experience or with realistic laboratory tests and in such cases the material requirements for particular applications can be assessed by Charpy tests.

The usefulness of the Charpy test and similar tests lies mainly in quality control. In various British Standards are specified minimum impact values for the given steels obtained from the results of Charpy V-notch tests. For example, B.S.968, the specification for high yield steel, requires that plates of this material up to and including 2in. thick are to produce minimum energy absorption values of 20 ft. lb. at −15°C (average of 3 tests). This figure represents merely what the manufacturer can achieve and does not relate in any way to 'real' structural behaviour.

Design considerations

The factors to be considered when designing against brittle fracture are listed below:

Applied stress level
Temperature
Material thickness
Rate of loading
Whether or not the structure is welded

and the question to be answered having established these is: What material is necessary to ensure freedom from brittle fracture for these conditions?

Structural steels available

The structural steels available in this country are covered by three specifications:

B.S.15: 1961, 'Mild steel for general structural purposes',
B.S.2762: 1965, 'Notch ductile steel for general structural purposes',
B.S.968: 1962, 'High yield stress (welding quality) structural steel'.

Steels to B.S.15 and B.S.2762 are both mild steels. B.S.15 is the usual mild steel which is used in the great majority of building and bridge structures. It is required to pass certain

Table 1: Required impact properties of steel to B.S. 2762

Grade	Test Temperature	Impact Properties Minimum Average (3 test pieces)	 Minimum individual test piece
	°C.	ft. lb.	ft. lb.
ND I	0	20	15
ND II	−15	20	15
ND III	−30	20	15
ND IV	−10	—	45
plates	−20	—	40
	−30	—	35
	−40	—	25
	−50	—	20
ND IV	−10	45	35
Sections	−20	40	30
	−30	35	25
	−40	25	20
	−50	20	15

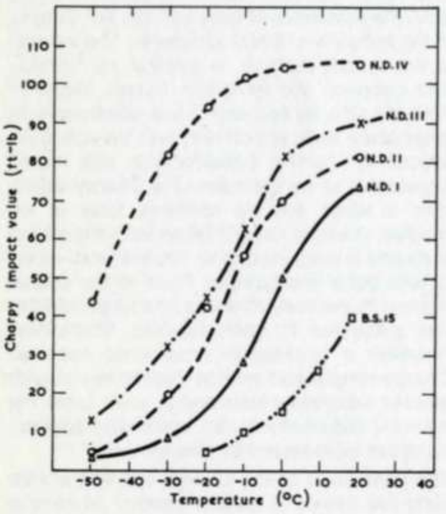


Fig. Comparison of impact properties of ND steels and steel to B.S.[1] (Illustrator: Margaret Woodard)

minimum strength and ductility standards, but no notch-impact tests are called for in its specification. The B.S.2762 steels were developed specifically to meet the need for mild steels whose resistance to brittle fracture is better than B.S.15. The term 'notch ductile' implies good ductility at the critical regions at the tips of cracks, or notches. The notch ductile steels are manufactured in two classes, A and B, with class B steels having the slightly higher minimum yield stress. There are four grades, ND I to ND IV, with progressively improving impact properties.

The impact properties of the ND steels are compared with those of B.S.15 in Fig. 9, which shows the results of Charpy V-notch tests. As a quality control the ND steels are required to show minimum Charpy impact properties at various temperatures and these are shown in Table 1.

Steel to B.S.968 is the only high yield stress structural steel now manufactured in the country. It was developed specifically as a relatively low cost, high strength steel which was readily weldable. It is required to have certain minimum impact properties in addition to the usual strength and ductility requirements. The impact properties required are shown in Table 2. Note that the manufacturers can achieve better impact properties for plates than for sections; this is because all plates of B.S.968 ½in. thick and over are normalised whereas sections are not.

These are the structural materials available. The next step in answering our question is to look to the British Standards for guidance in the use of these materials.

Help from British Standards

Enough information is now available in the standards to enable this question to be answered for the great majority of practical cases. Two standards are available which between them cover the use of the steels in buildings and in bridge construction. They are:

B.S.449:1959, 'The use of structural steel in building' (Amendment No. 6 (1966)), and B.S.153, Parts 1 and 2, 1958 (1966 reprint), 'Steel girder bridges'.

These standards limit the thickness of material which may be used as welded tension members in a structure. They apply for temperatures down to −7 °C which is taken as the lowest design temperature in the United Kingdom. B.S.449 limits the material thicknesses as follows:

B.S.15	1½ in.
B.S.2762 All grades	2 in.
B.S.968 Plates & sections	2 in.

Table 2: Required impact properties of steel to B.S. 968

Plates	
Up to and including 2 in. (50.8 mm) thick	Average not less than 20 ft. lb. (2.76 kgm) at −15 °C.
Over 2 in. thick.	These values to be obtained shall be agreed between the manufacturers and the purchaser.
Sections & Flats	
Up to and including ¾ in. (19 mm) thick	Average not less than: (i) 20 ft. lb at 0 °C for standard test pieces, or (ii) 15 ft. lb. at 0 °C for subsidiary standard test pieces.
Bars—Rounds, square, etc.	
Up to and including 1⅛ in. thickness or diameter.	

There is a provision for greater thickness of B.S.2762. For such thicknesses the impact test requirement must not be less than 20 ft. lb. at 0 °C. Further impact test requirements are called for under certain conditions such as dynamic or shock loading or if the ambient temperature falls below −7 °C. The maximum thicknesses imposed in B.S.153 are more stringent and this is because bridges may be subjected to more severe conditions than buildings. This covers fluctuating loads, exposure to climatic conditions and some degree of impact loading. The allowable thicknesses are:

B.S.15		¾ in.
B.S.2762 Grade ND I		1½ in.
B.S.2762 Grades ND II, ND III, ND IV		2 in.
B.S.968	Impact test not stipulated:	
	Channels	½ in.
	Plates & normalised flats	1¼ in.
	All other forms	¾ in.
B.S.968	Impact test stipulated	2 in.

B.S.153 also states:

For thicknesses greater than 2 in., and for service in countries experiencing minimum atmospheric temperatures below those of the U.K. particular attention shall be paid to the selection of steel with adequate notch ductility which will be a matter for agreement between the manufacturer and the purchaser.

This last paragraph poses the question of how to select a steel with adequate notch ductility. Tests are available from which the notch ductility of a steel in the anticipated service conditions may be determined. The tests are of a specialised nature and will only be mentioned here. However, the philosophies behind them are described as they explain the current thinking in brittle fracture research.

Help from special tests

There are two basic approaches to ensure freedom from brittle fracture. They both allow that flaws in the welds are inevitable and hence that starting points for brittle cracks are inherent to the weld regions. They are:

1.
Preventing initiation of cracks from existing flaws.

2.
Preventing propagation of cracks after initiation.

The material in the critical regions at the tips of flaws will have differing transition temperatures. In order to *prevent initiation* of cracks from these flaws, it is essential to know the transition temperature for the 'worst case'. This can be found by carrying out tests on the material from the weld area and the parent plate. The particular tests used for this are either a technique simulating a full-scale welded structure, the British Welding Research Association (BWRA), 'Wide Plate' test (1), or a more recent technique known as the crack opening displacement (COD) test (3). Having determined the transition temperature of the worst case, then the steel can be selected which has a transition temperature above this so that critical regions at each flaw tip will yield plastically.

To *prevent propagation* the best approach is to ensure that the material is sufficiently tough to prevent the crack extending into the parent material from the weld. The material has to be able to absorb the energy released by a moving crack tip, that is, at high strain rate conditions. The transition temperature for parent plate for static (i.e. initiation) conditions is very much lower than that for dynamic (i.e. propogation) conditions. A test currently in use to determine the transition temperature of a steel under dynamic loading is the Robertson test. (1).

Of the two approaches, the second is the safest. Also it does not involve the testing of various critical parts of the weld zone, tests on the parent plate only are necessary.

Table 3: Tentative minimum temperatures to which structural steels may be used for static loading conditions (2).

Steel	Thickness (in.)	*Minimum Temperature °C* As welded	Stress relieved	Charpy requirements for minimum temp. as welded (ft. lb.)
B.S.15	1/4	−30	−100	5–15
	3/8	−25	− 95	
	1	0	− 75	15-25
	2	+10	− 55	
ND I	1/4	−40	−100	5-15
	3/8	−30	− 95	
	1	− 5	− 75	15-25
	2	0	− 55	
ND II	1/4	−50	−100	5-15
	3/8	−35	− 95	
	1	−15	− 75	15-25
	2	− 5	− 55	
ND III	1/4	−65	−100	10-15
	3/8	−40	− 95	
	1	−20	− 75	20-30
	2	−10	− 55	
ND IV	1/4	−90	−100	10-15
	3/8	−60	−100	
	1	−25	− 80	25-35
	2	−15	− 60	
B.S.968	1/4	−40		10-20
	3/8	−30		
	1	−15		25-45
	2	− 5		

However, steels which have a high toughness at relatively low temperatures are fairly costly and it is not always possible to ensure adequate toughness at the working temperature. Since commonly used structural steels such as B.S.15 and B.S.968 may have adequate toughness in the region of 0– +30°C for thicknesses above 1 in., usage of these steels for normal structural work in cold climates must rely upon prevention of initiation.

Summary and discussion

A steel subjected to a tensile load which fails below its transition temperature will always fail along the cleavage crystal planes and not along its shear planes but this need not be a brittle fracture in the sense described in this paper. A brittle fracture implies low applied stresses with little or no ductility surrounding the fracture surfaces and for this type of behaviour other conditions are necessary but not always sufficient for a brittle fracture to occur. The propensity of a steel to brittle fracture is influenced by material thickness and the nature of the loading (whether impact or static) in addition to working temperature and applied stress level, but the overriding factor is the presence of welding. Because welding introduces residual stresses and notches, and raises the transition temperature of the material at the weld region, welded joints are critical regions for the initiation of brittle cracks. A crack once started can then travel away from the weld through material which would be too tough to initiate such a crack.

For design against brittle fracture guidance is given in B.S.449 and B.S.153 for statically loaded welded structures and welded girder bridges respectively, for working temperatures down to −7°C, but this should be regarded as a minimum and not as a guarantee of freedom from trouble in all circumstances.

The British Welding Research Association has recently suggested minimum temperatures to which structural steels may be used at different thicknesses for welded tension members under static loading, with reasonable design, fabrication and inspection standards (2). This information is reproduced in Table 3. The table would apply to steel-framed buildings. Note that these restrictions on material thickness are more severe than those of B.S.449. For example, whereas B.S.449 allows the use of B.S.15 in thicknesses up to 1½ in. for temperatures down to −7°C, Table 3 would limit this to less than 1 in.. It is prudent to follow Table 3 rather than B.S.449 in such a case.

The benefits to be derived from stress relieving are clearly seen. No information is at present available on stress relieved B.S.968, but it can be compared with B.S.15. The temperatures shown for the stress relieved material can be considered suitable for non-welded structures under static loads. In principle Table 3 applies to plate material only; however the minimum temperatures are based upon the Charpy impact test requirements shown and if these can be achieved for sections the table may be used for these also, for B.S.15 and the ND steels. The main difference between plates and sections with regard to the Charpy levels arises with B.S.968. For dynamic loading, higher Charpy values, of the order of 40 to 50 ft. lb. at the minimum working temperature, should be called for. The safest course is to ensure that the steel is working above its transition temperature for dynamic conditions and for this expert advice is advisable.

Good design details can help. In the same way that notches act as stress raisers so do changes of section, and if a sharp change of section is present in the region of a weld subject to tensile stresses, as for example, the end of a welded cover plate on the tension flange of a girder, a dangerous situation may be created for the initiation of a brittle crack. The fatigue clauses of B.S.153 indicate the lives which can be expected from different welded details under given stressing conditions. Whilst this information applies to fatigue loading one can infer from it the inherent stress concentration effects of different types of welded joint. Just as improving the detailed design will help prevent fatigue failures so a similar approach will avoid brittle fractures.

It must be pointed out, however, that whilst good design detailing can help, material selection is the more important since this is necessary to cater for the presence of defects associated with the welded joints. For a structure such as a bridge or gantry it may be better to rely upon tougher steels to provide adequate safety rather than upon finesse in design or fabrication. The structure may suffer change later on in the course of routine maintenance or changing duty and this work may be carried out by people ignorant of the brittle fracture problem.

Material selection is often a matter of economics and the decision whether to use B.S.15, the ND steels or B.S.968 may be determined by cost and delivery dates rather than for any technical reason. It is often good practice to make the main tension flanges of heavy plate girders and other elements judged to be vital, of ND I, but alternatives are either to use B.S.15 and provide a 'crack barrier' of ND IV or to stress relieve locally (with expert help). The decision may again depend upon economics.

Thermal stress relief of large fabrications is usually prohibitively costly but it is sometimes worthwhile to stress relieve small welded components such as connecting cleats, which are to be bolted onto the main structure. It should be borne in mind that such things as cleats and end plates which are welded onto an otherwise bolted structure cause residual stresses, cracks, etc. in the main structural members to which they are attached and hence provide the critical regions in such structures.

It has been assumed throughout that the welding has been carried out satisfactorily and that the requirements laid down in B.S.1856 and B.S.2642 for the welding of mild steel and high yield steel respectively have been followed. Extreme care is essential when welding B.S.968 steel in the heavier sections.

Finally if a problem arises which lies outside the scope of the standards or if difficult decisions have to be made concerning material selection, stress relief, use of normalised material etc., then it is advisable to seek expert advice. In this country the best advice available is from the BWRA. We as a firm are members of this Association and hence can obtain advice on such matters. The Association provides a design advisory service in addition to having a section concerned entirely with brittle fracture. Any testing which may be necessary can be carried out by the Association, on a contract basis.

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