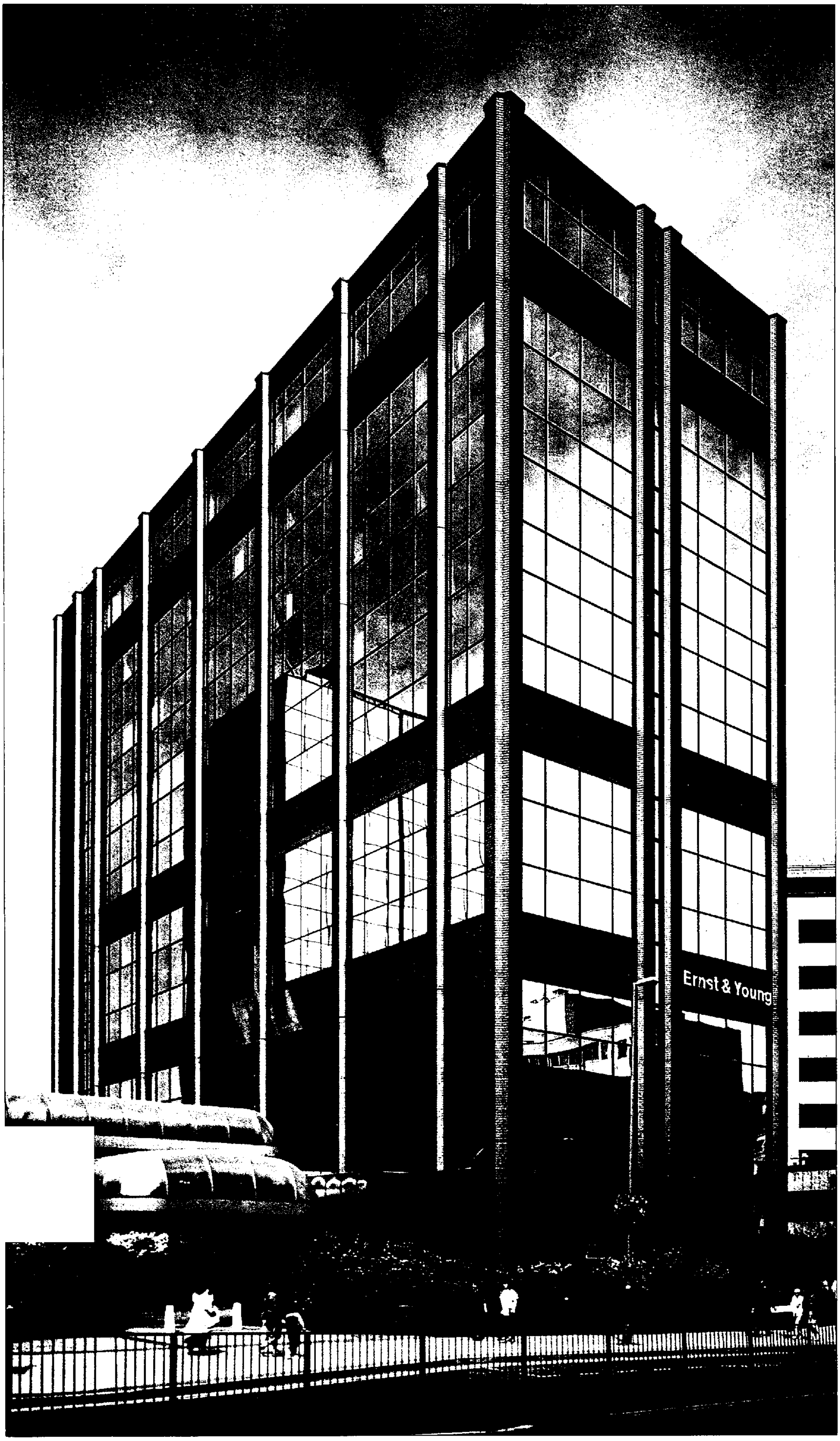
PRECAST CONCRETE FRAME BUILDINGS



DESIGN GUIDE

K.S. ELLIOTT
A.K. TOVEY

BCA

■ ■ Many construction activities are potentially dangerous so care is needed at all times. Current legislation requires all persons to consider the effects of their actions or lack of action on the health and safety of themselves and others. Advice on safety legislation may be obtained from any of the area offices of the Health and Safety Executive.

47.024
First published 1992
Reprinted with minor amendment 1992

ISBN 0 7210 1416 X

Price Group K

Published by
British Cement Association
Telford Avenue
Crowthorne
Berkshire
RG11 6YS

Telephone (0344) 762676
Fax (0344) 761214

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Designed and printed by The Ethedo Press Limited, High Wycombe, Buckinghamshire.

PRECAST CONCRETE FRAME BUILDINGS

DESIGN GUIDE

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INTRODUCTION

Precast concrete frames are well established for the construction of low-rise and multi-storey offices and for elevated car parks. There are also many examples of precast concrete frames in retail, industrial and warehousing developments.

Their record of success is such that a precast concrete frame should always be included when alternative methods of construction for any new building project are being assessed. Without this, clients or their professional advisers may well miss the significant benefits which precast concrete frames have to offer.

The problem has been that there is a wealth of general and detailed information on many structural forms, but surprisingly little to help engineers and architects to achieve a full understanding of precast concrete building structures and their procurement.

This publication is intended to fill this gap by providing a detailed review of the subject and thereby promoting a greater awareness and understanding of precast concrete frame buildings. It has been written particularly for those less familiar with this form of construction, but will also be of interest to all engineers, architects and others concerned with the procurement of buildings.

Kim Elliott is a Lecturer at Nottingham University in the Department of Civil Engineering, where he has supervised research into various aspects of precast concrete frames. Previously he worked for a major precast concrete manufacturer.

Alan Tovey is Associate Director, Building and Structures, within the Technical Marketing Division of the British Cement Association, with particular responsibility for precast concrete. Before that he was involved in the design and construction of precast concrete buildings for one of the largest precast concrete manufacturers in the UK.

ACKNOWLEDGEMENTS

The authors are grateful for comments from members of the Structural Precast Concrete User Group and for the information and comments from members of the Precast Concrete Frame Association. The authors also thank all those who supplied photographs and other material.

In particular their thanks go to Andrew Dyson, Building Design Partnership, for his contribution to Section 2, *Procurement*, and the Appendix; also to John Richardson, Consultant, for his input to Section 5, *Production*. Thanks are also extended to all those who have assisted with the production of this publication, especially to Dennis Palmer for his considerable effort as BCA's Publications Co-ordinator.

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5 PRODUCTION

6 STRUCTURAL DESIGN

PRECAST CONCRETE FRAMES

AN OVERVIEW

Quality and accuracy

Precast concrete units are made in a factory in a favourable environment and with tight production control. This produces units with high quality performance and appearance. The designer can select from a range of finishes and be able to inspect and accept the units before they are fixed in place.

Factory production control ensures that the reinforcement is located accurately and that the units are made to tight dimensional tolerances. The structural connections are designed to enable adjustments to be made on site, so the frame can be erected to very precise dimensions. This greatly assists the subsequent installation of cladding, windows and other elements.

Speed of construction

Speed of construction is a major consideration in most building projects and it is here that precast concrete frames excel. This advantage is maximized if the layout and details are not too complex. Designing for maximum repetition will make manufacture of the precast units easier and construction faster, but precast concrete can be used in complex and irregular structures, although it may not then provide the same efficiency of construction as a rationalized design.

Precast concrete frames can increase the overall speed of construction by allowing parts of the structure to be released to following trades whilst work continues on erecting the rest of the frame.

The preferred method of erection is to construct bays to the full height whilst backing out of the building, enabling the frame to be released bay by bay. An alternative method is to erect the frame floor by floor, thus releasing lower floors whilst work continues on the erection of the floors above. Access can often be gained within two or three weeks of starting to erect the frame.

Frame cost

In addition to the economic advantages of faster construction, the capital cost of a precast concrete frame can be less than that for alternative framing methods designed to give an equivalent performance.

Alternative frames for a five-storey commercial building have been compared.(1) These showed that the cost of a precast concrete frame was 21% less than a steel frame with steel deck composite floors, and 14% less than a more traditional steel frame with precast concrete floors. Another comparison on a higher, seven-storey structure, suggested a similar saving.(2) These savings may not be realized on all projects, because the difference will be influenced by the particular design parameters, but they demonstrate a potential cost saving which is worth considering.

Overall cost

The total cost of a building is related to the speed of construction, the cost of the frame, other construction costs, land costs and interest rates. Although the basic cost of the frame is important, speed of construction is often the dominant consideration, particularly in times of high interest rates and land values. Where the building is to be let, extra rental can be obtained from earlier completion.

Thermal capacity

The high thermal capacity of a concrete structure can help to control temperature fluctuations. This can reduce the risk of condensation. Peak demands on the heating, ventilating and air conditioning equipment may also be reduced and lead to cheaper service installations and lower running costs.

Buildability

Precast concrete frames can greatly improve buildability. Compared with many other methods of construction, precasting removes many of the sensitive site operations to the more stable environment of the factory. Bad weather has little effect on the rate of frame construction and little protection is needed on site.

Precast concrete frames are precisely manufactured to improve speed of erection. The care taken in design and detailing of the connections ensures that erection is simple and rapid, and structural integrity is achieved during erection. Crane hook time is kept to a minimum.

The precast concrete supplier, as a single subcontractor, is usually responsible for the design, production and erection of the frame. Minimizing the number of subcontractors simplifies contract programming and can reduce pressures on the management team. Contract periods may also be forecast more confidently because, with fewer operations, there is less to go wrong.

Structural efficiency

Precast concrete offers considerable scope for improving structural efficiency. Longer spans and shallower construction depths can be obtained by using prestressed concrete for beams and floors. The examples of buildings given later in this Section and elsewhere in this publication, demonstrate the considerable flexibility that is possible in design. Many of the available precast concrete units have been tested, both in the laboratory and in service, to obtain maximum structural efficiency.

Fire resistance
Concrete has its own built-in fire resistance. This is present during all construction phases and does not depend on additional board or sprayed protection. Obtaining a two-hour fire resistance presents little problem, and four hours may also be achieved.

Design principle
Most projects benefit from disciplined design. This is inherent in the design of a precast concrete frame, because the designer seeks as much repetition as possible so that the precaster can take advantage of greater mould re-use and standardization of details to reduce the cost of manufacture.

Design flexibility
Modern precast concrete frame buildings can be designed safely and economically, with a variety of plans and with considerable variation in the treatment of the elevations, to heights up to about 50 metres. The variety of possible designs is indicated in the descriptions of UK projects which follow.

PROJECT EXAMPLES

Borough Council Offices, Brighton

Supermarket, Kings Road, Brentwood

Swan Office Centre, Birmingham

Friary House, Southampton

Woodchester House and Merchant House, City Harbour, London Docklands

Snow Hill Redevelopment, Birmingham

Capital House, Edinburgh

Huntavia House, Longford, Near Heathrow

Horsely Down Square, London

Watchmoor Park, Camberley

Sunbury Business Centre

Daily Mail, Surrey Docks, London

Imperium, Reading

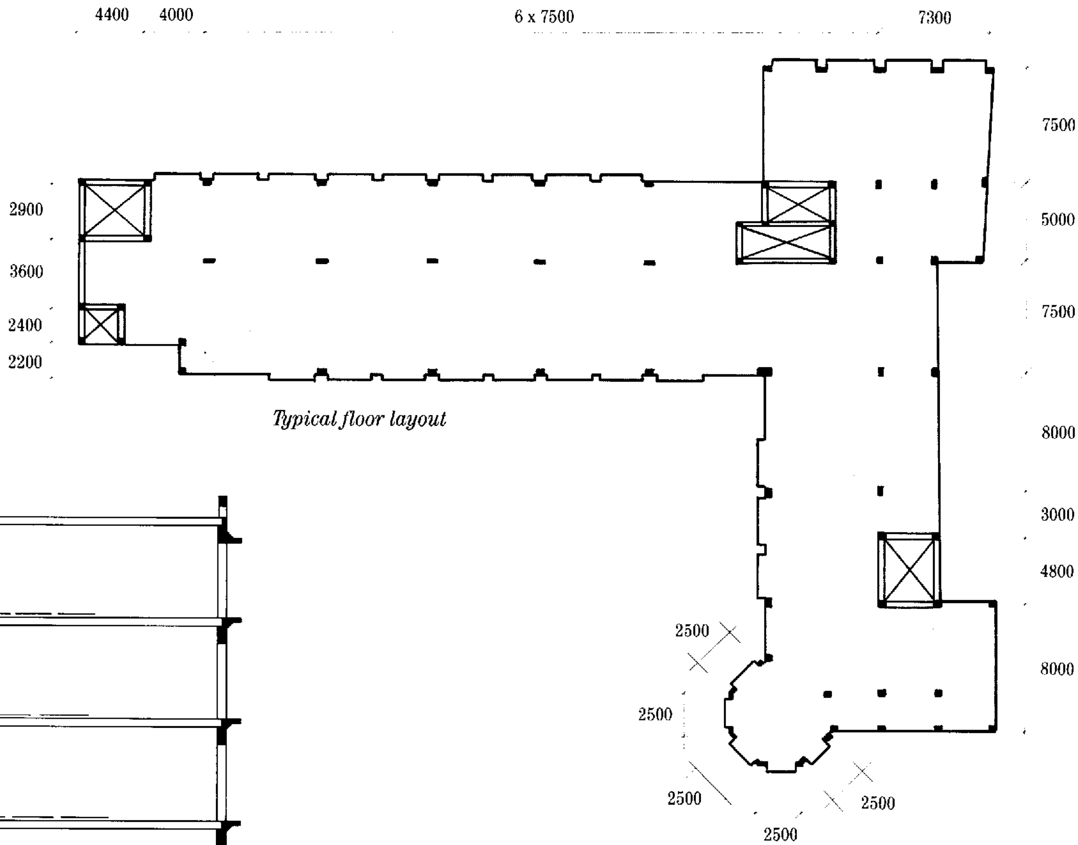
The Shires, Leicester

Watermead, Aylesbury

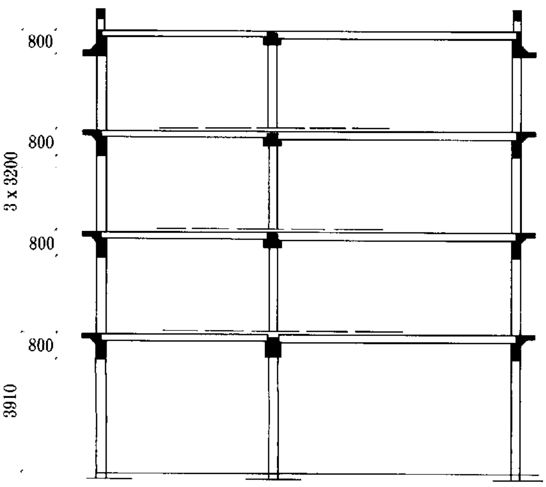
The Waterfront, Brierley Hill, Dudley

BOROUGH COUNCIL OFFICES, BRIGHTON

- Frame with high quality visual concrete.
- Octagonal end frame.
- Cantilevered beams to support cladding.

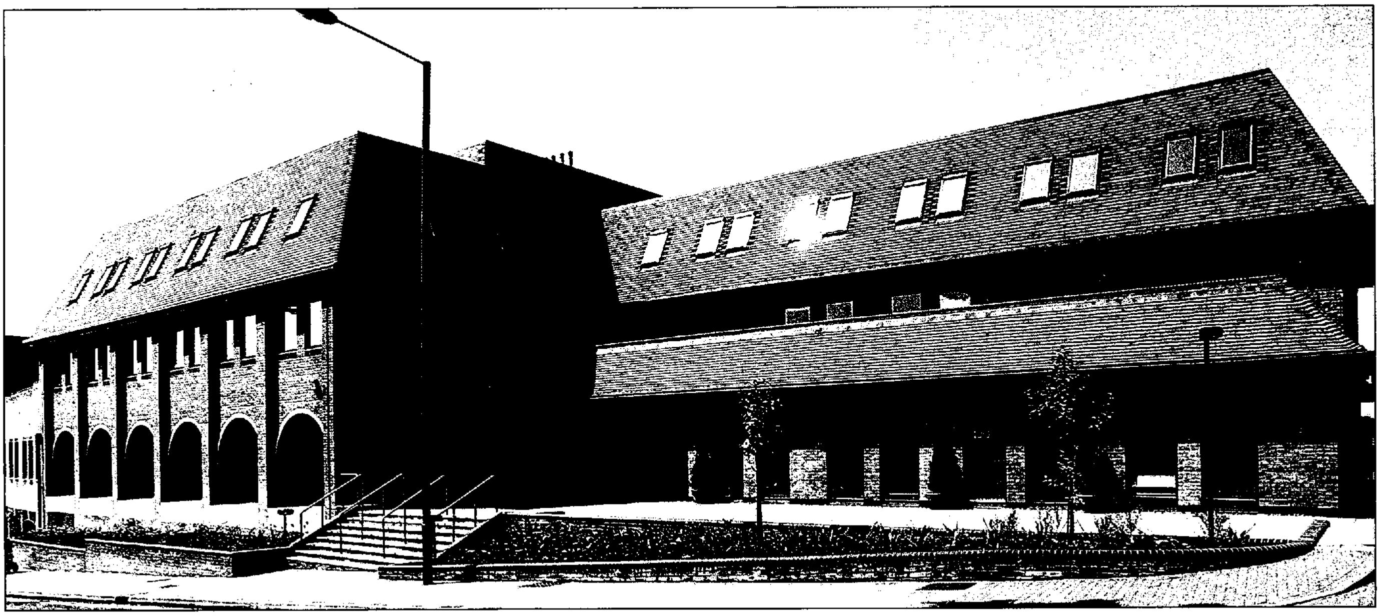


Typical floor layout

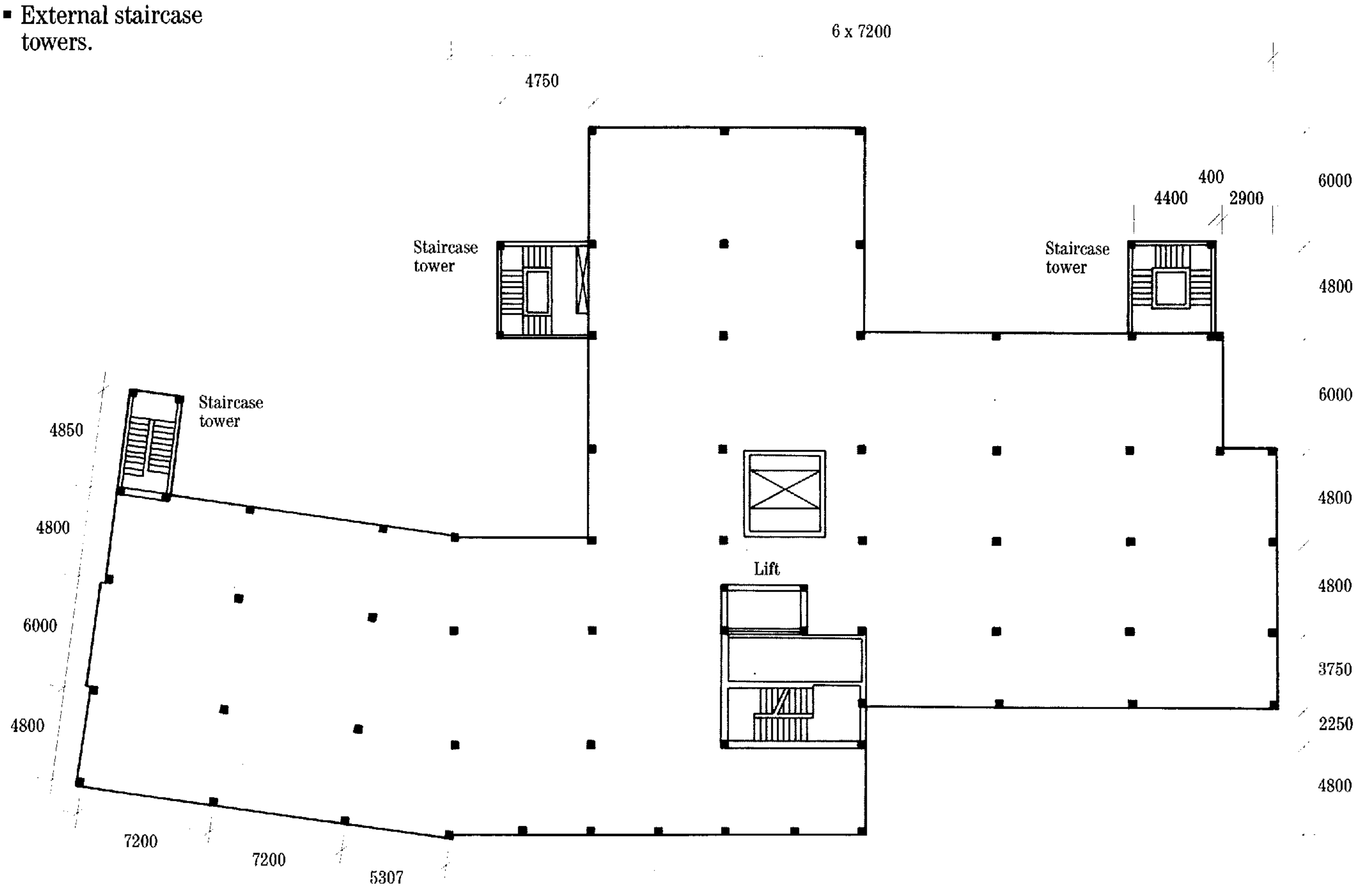


Typical cross section

■ SUPERMARKET, KINGS ROAD, BRENTWOOD



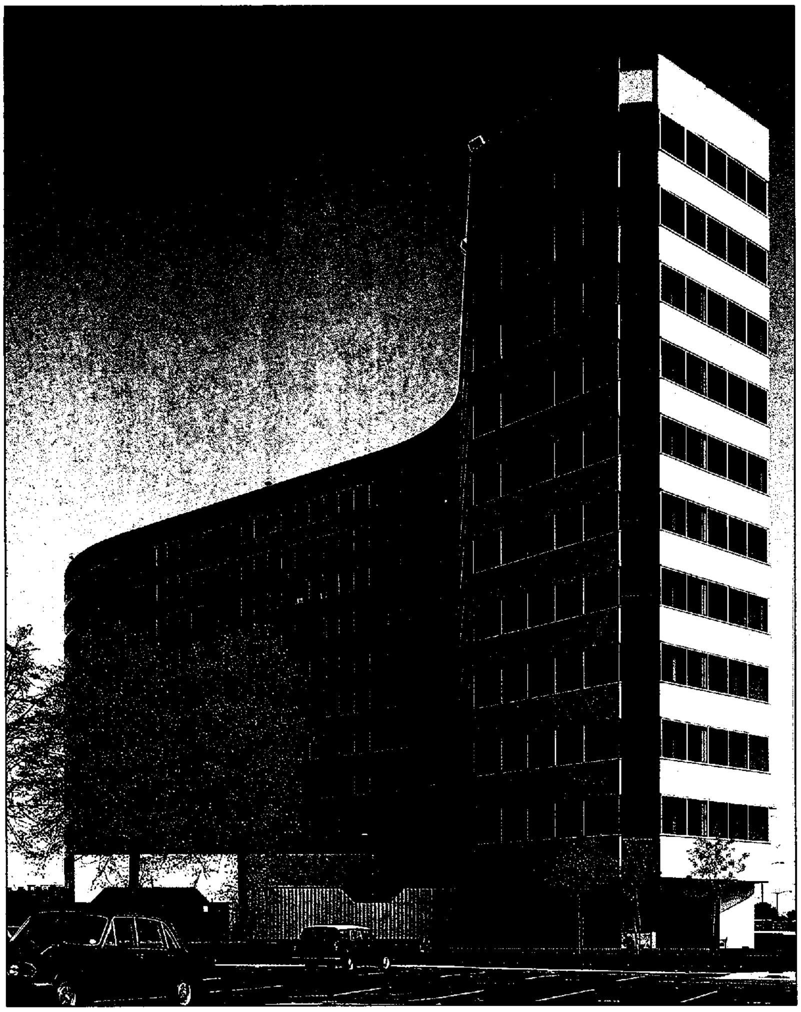
- Irregular plan building.
- Arch-shaped beams demonstrating concrete's mouldability.
- External staircase towers.

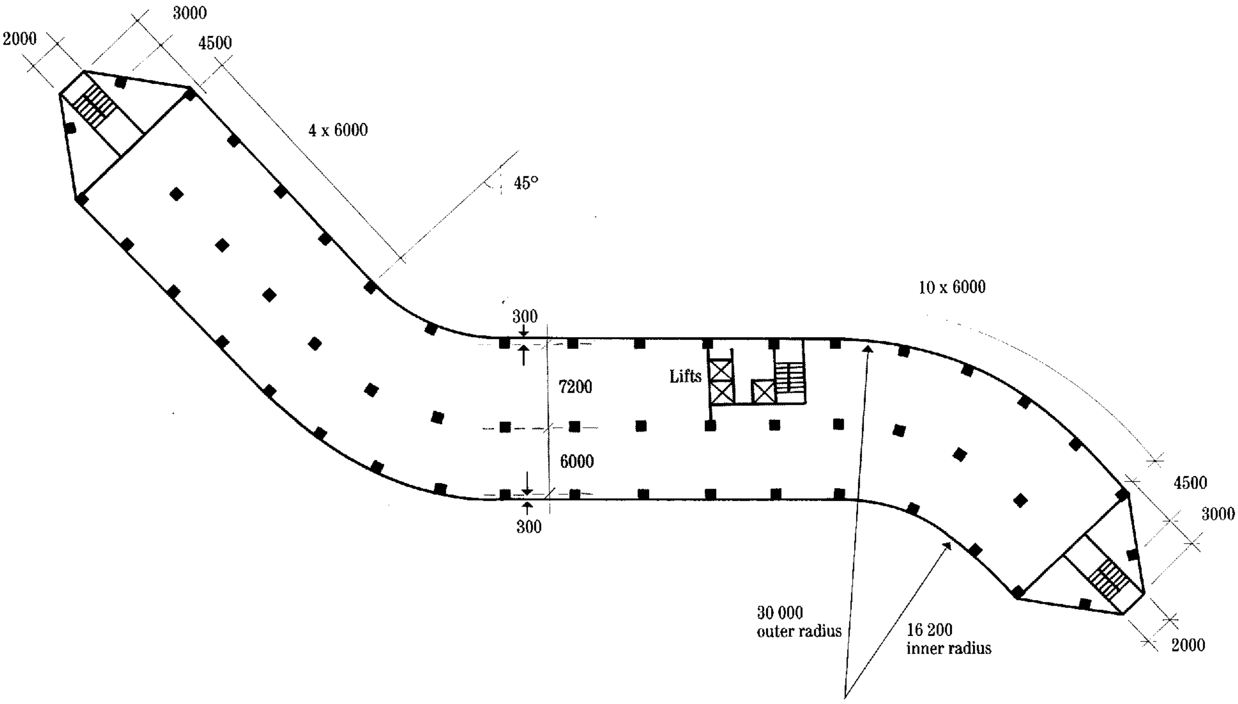


Typical floor layout

SWAN OFFICE CENTRE, BIRMINGHAM

- An eleven-storey structure.
- Curved plan form and load-bearing spandrel edge beams.

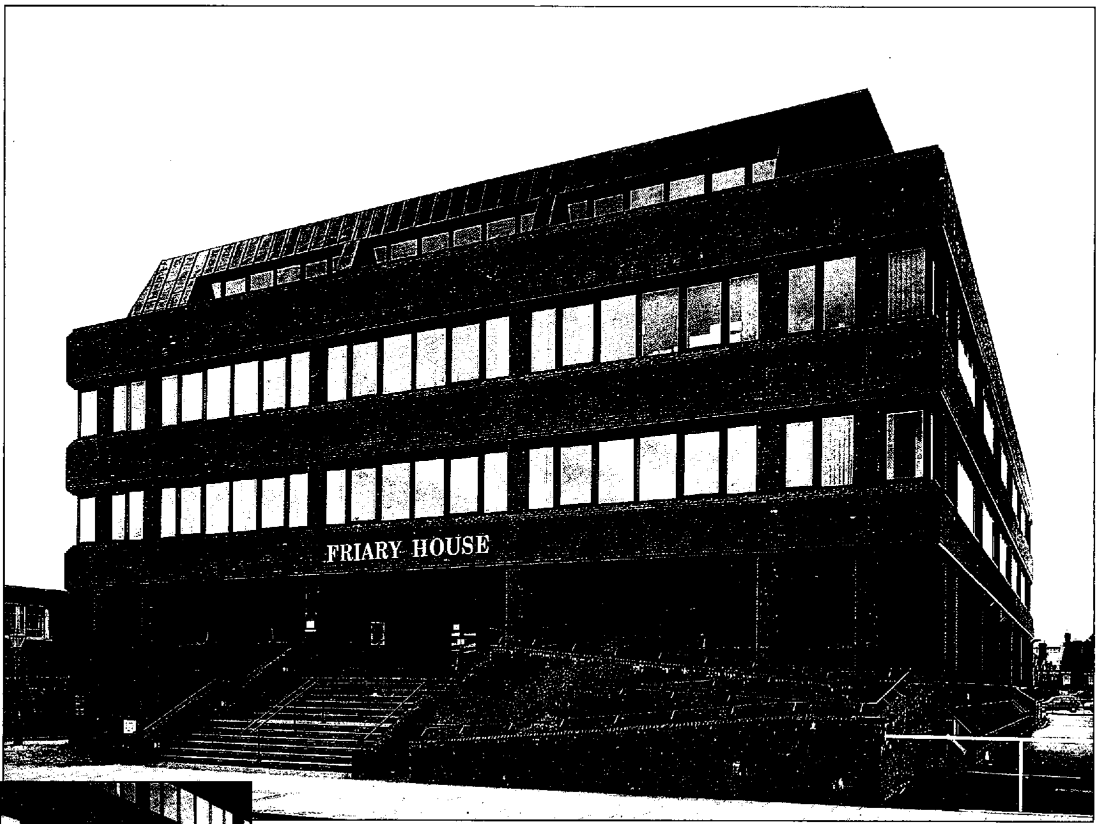


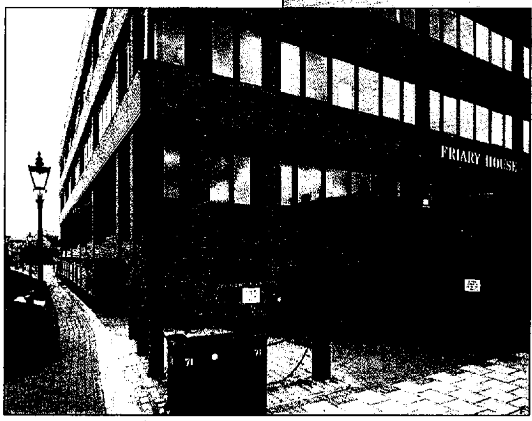


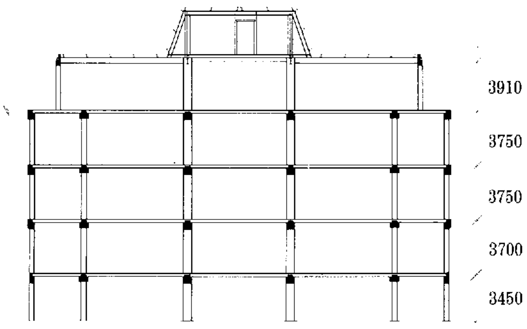
Typical floor layout

FRIARY HOUSE, SOUTHAMPTON

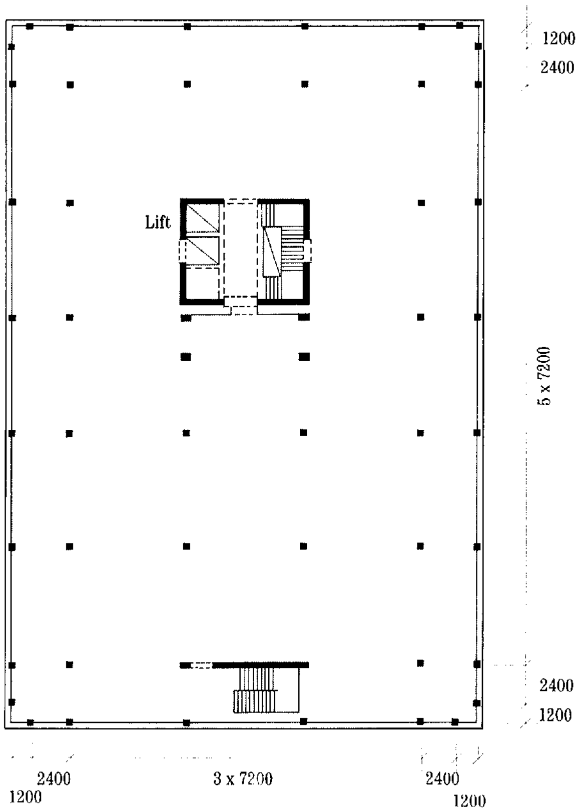
- Underground car parking.
- Perimeter frame set back off grid at floor four.
- Cantilever corner beams.







Typical cross section



Typical floor layout

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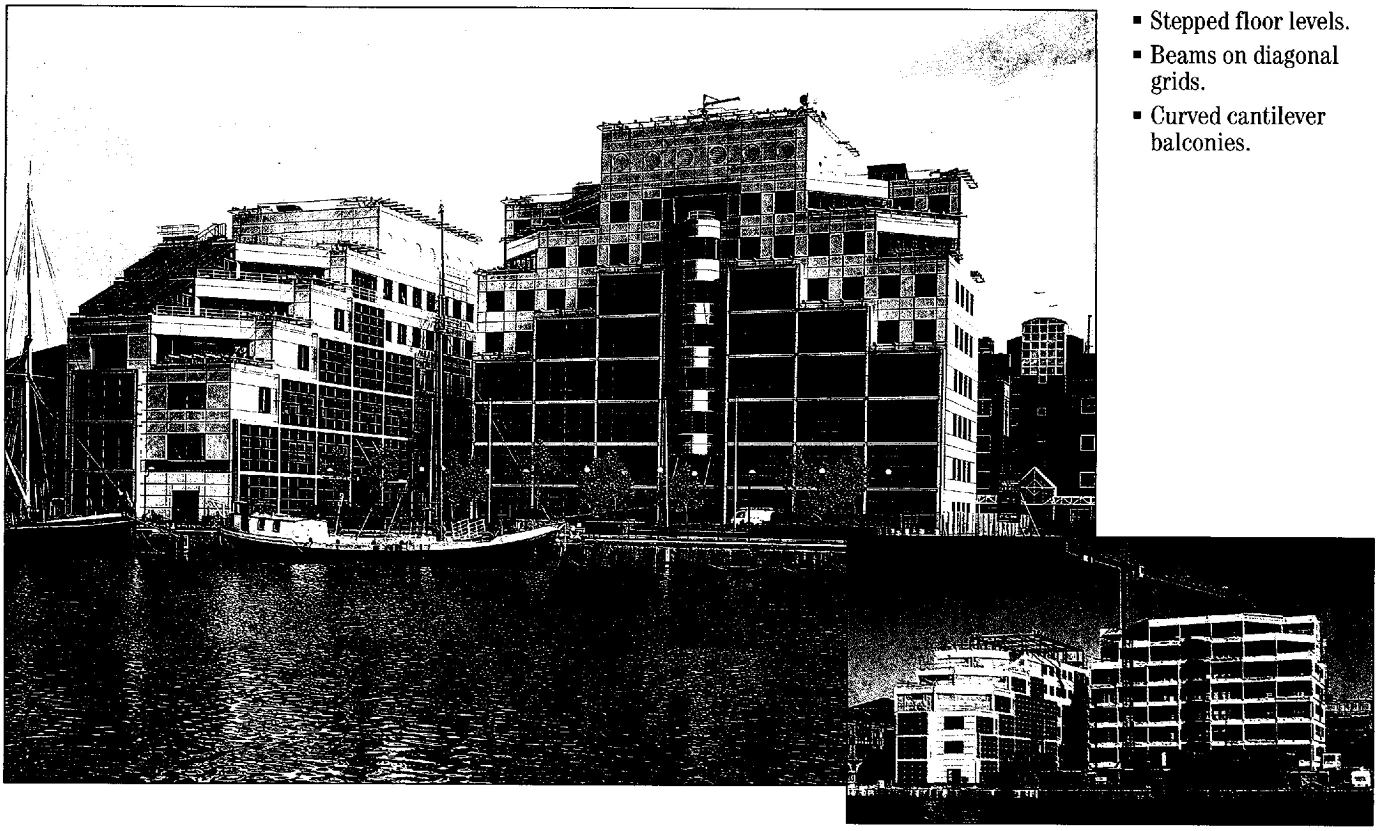
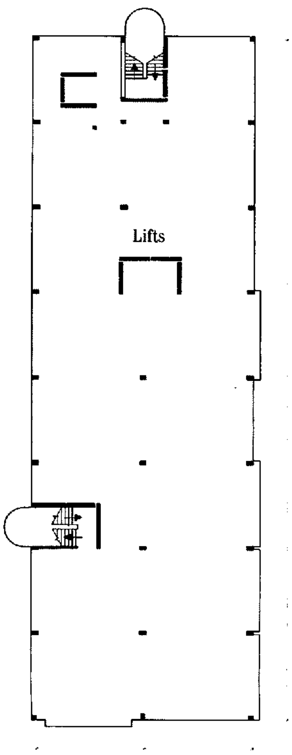
■ WOODCHESTER HOUSE AND MERCHANT HOUSE, CITY HARBOUR, LONDON DOCKLANDS

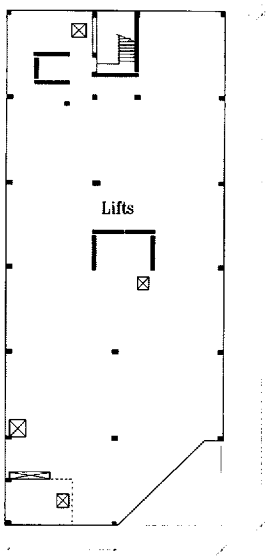


- Stepped floor levels.
- Beams on diagonal grids.
- Curved cantilever balconies.

Woodchester House

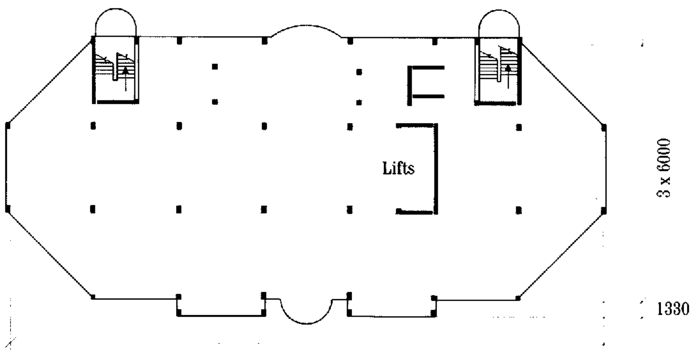


Layout of floor one

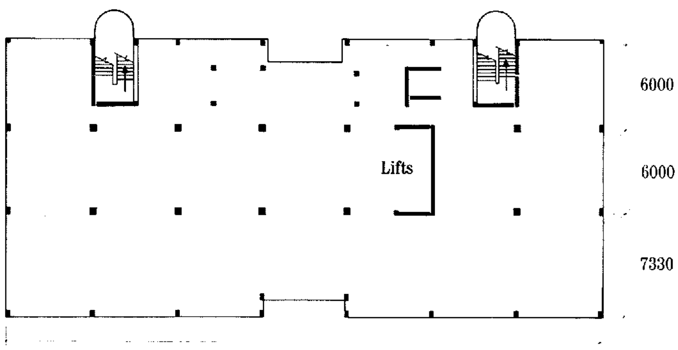


Layout of floor six

Merchant House



Layout of floor seven



Layout of floor one

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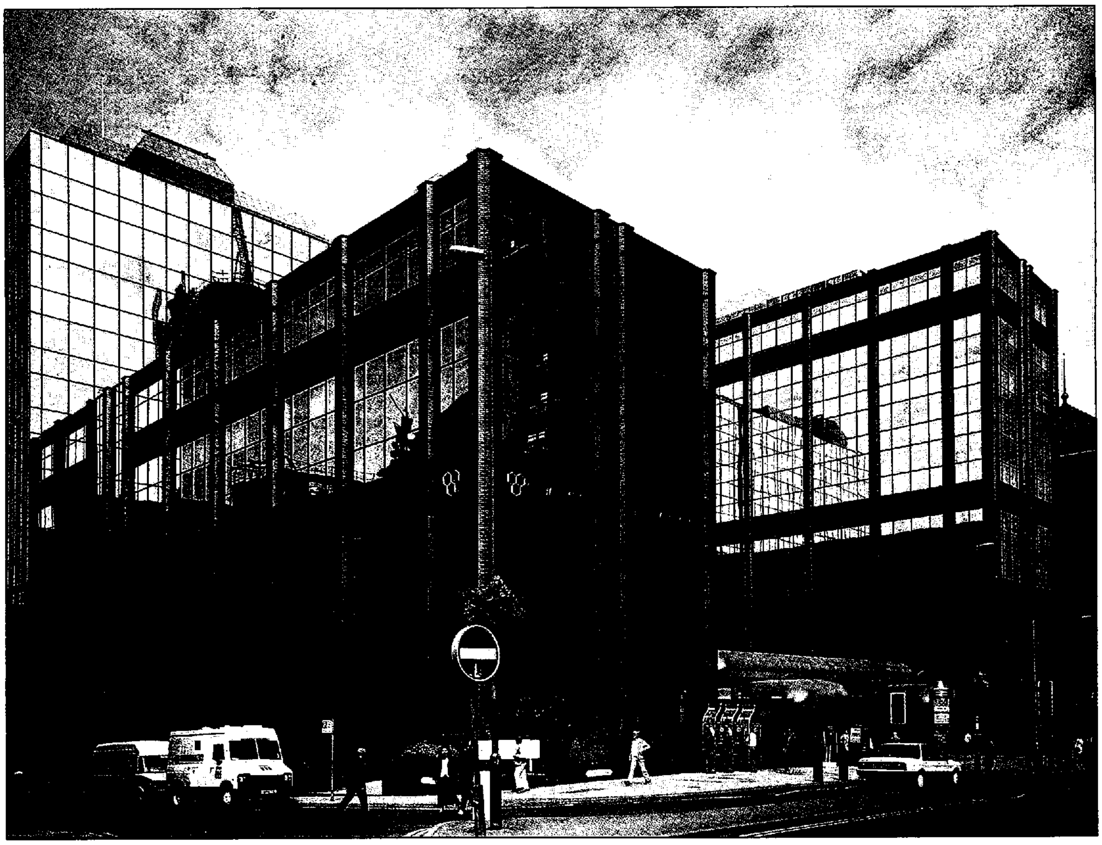
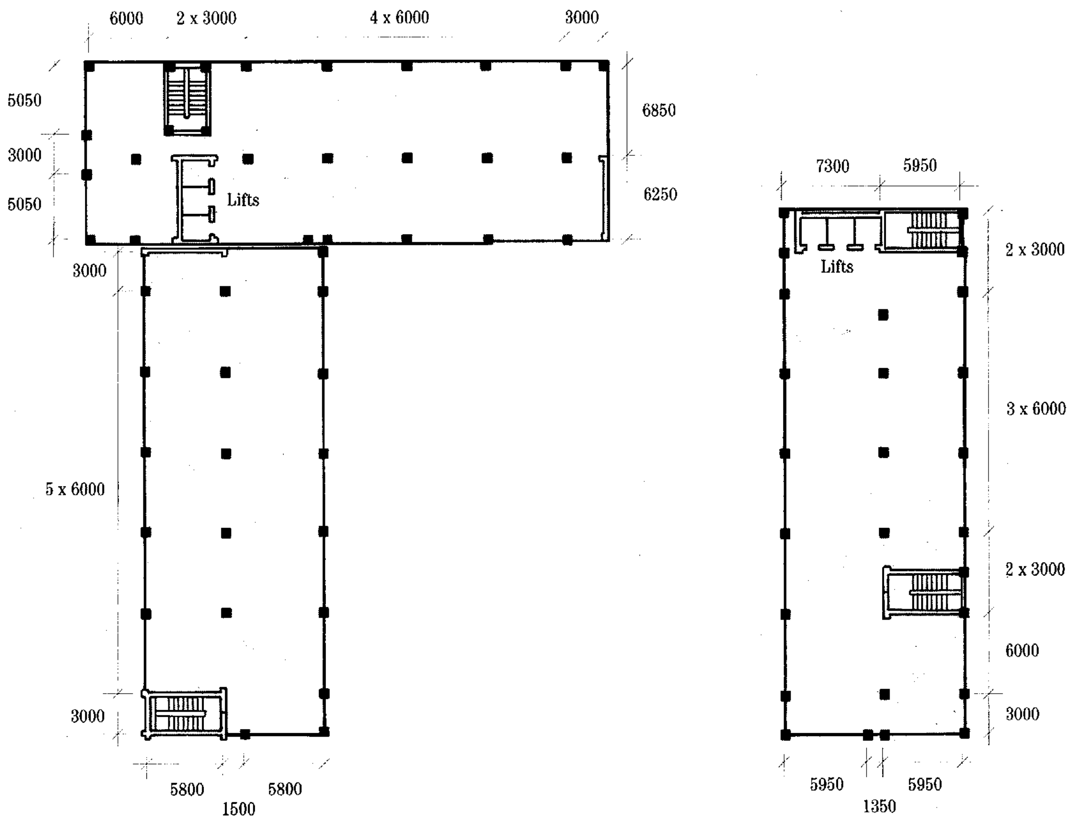
SNOW HILL REDEVELOPMENT, BIRMINGHAM

- A three-unit development.
- Two twelve-storey blocks plus an eight-storey wing.

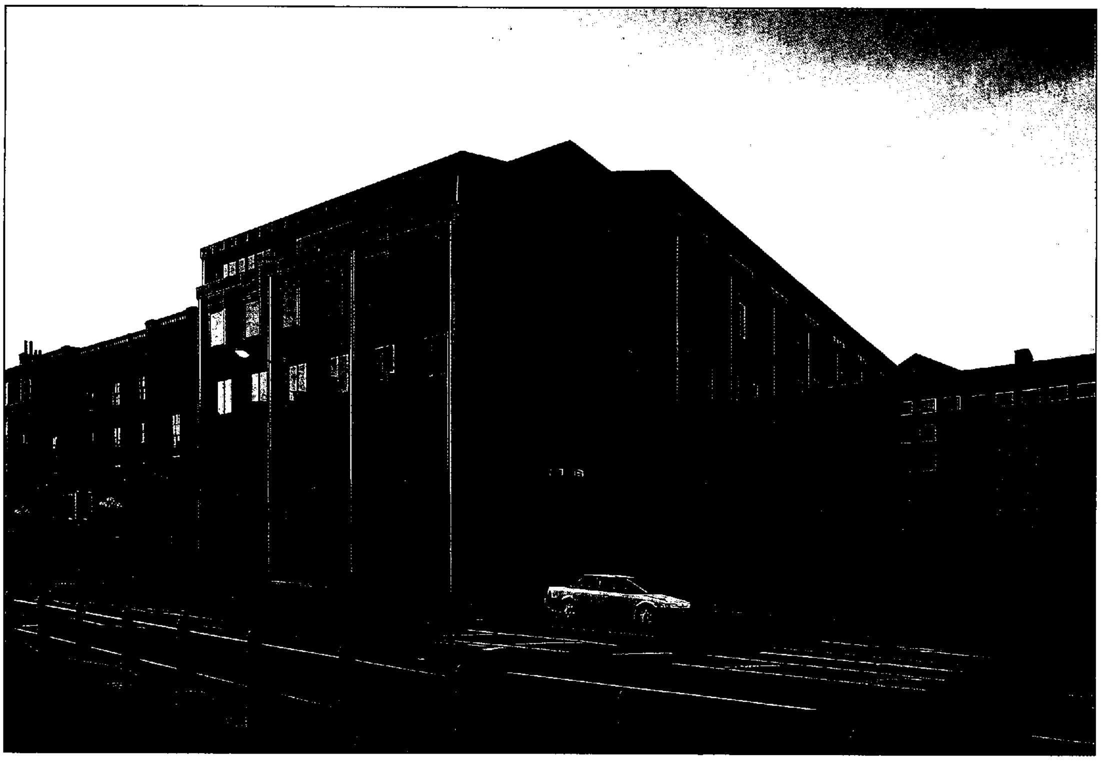


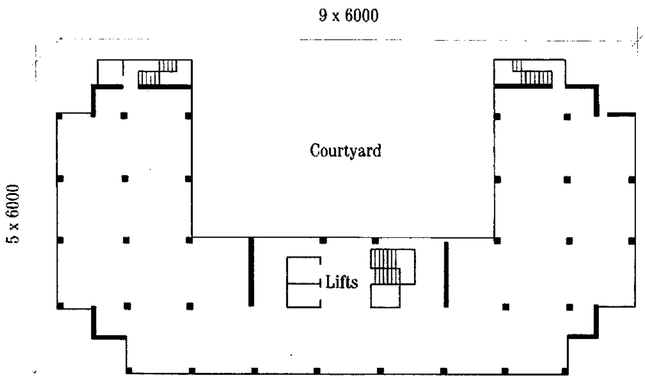


Typical floor layout

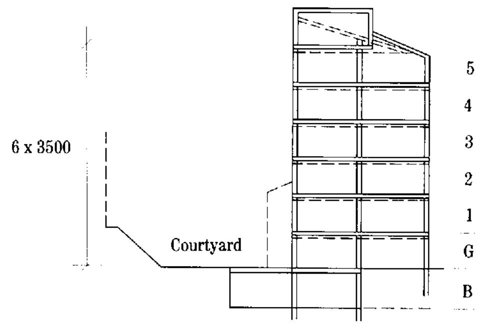
■ CAPITAL HOUSE, EDINBURGH

- A frame with a stepped plan.
- Infill panels used to create window openings.

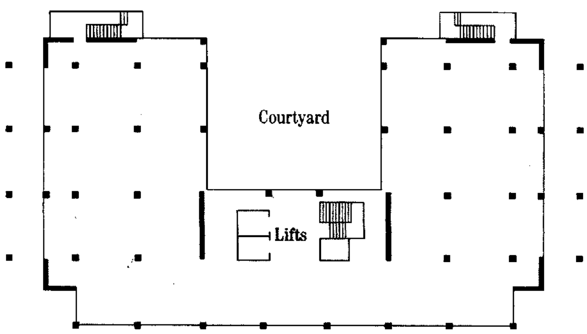




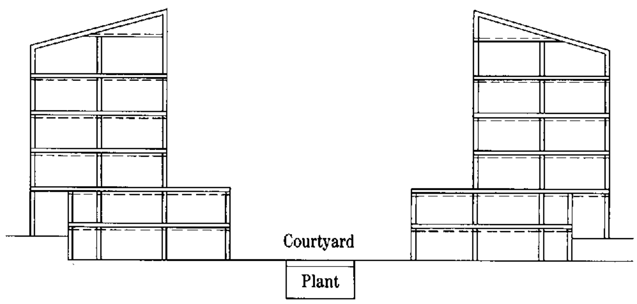
Layout for floors two to five



Section through north wing



Layout for ground and first floors

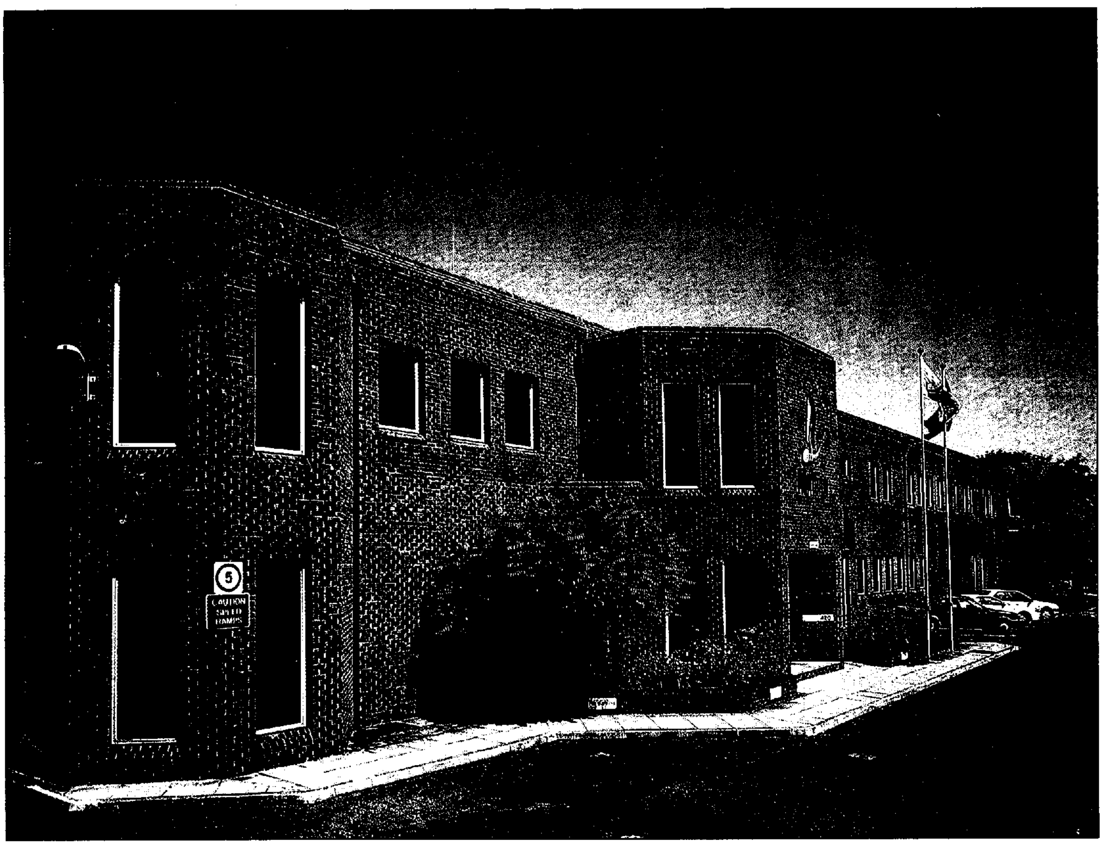


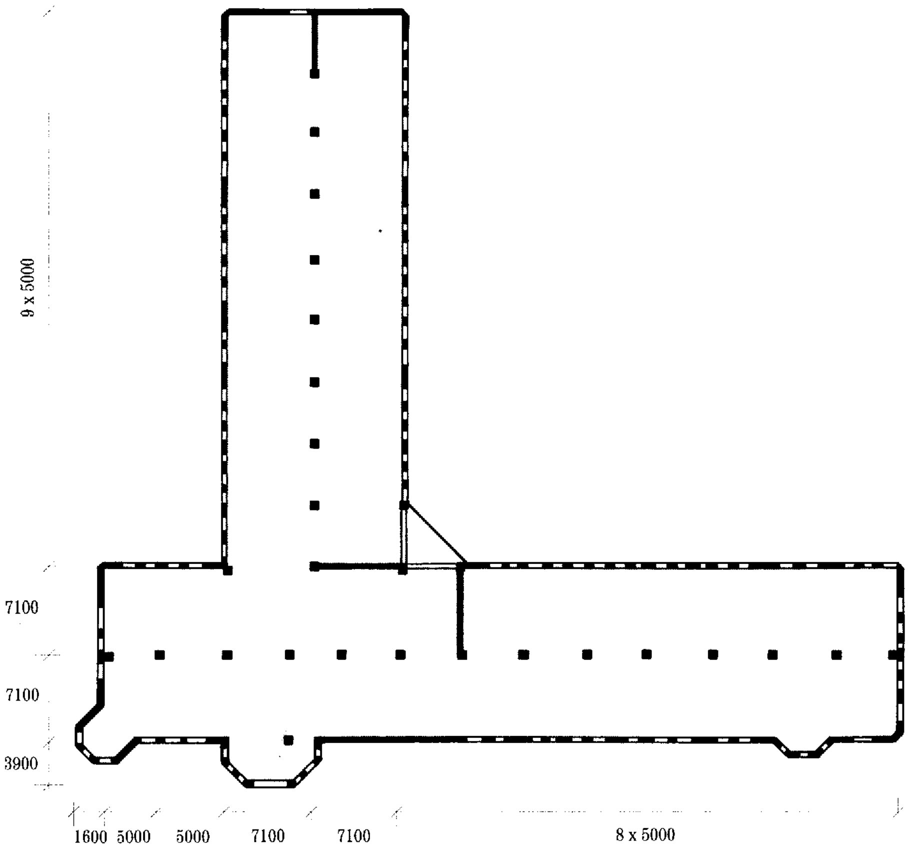
Section through west and east wings

51

■ HUNTAVIA HOUSE, LONGFORD, NEAR HEATHROW

- Loadbearing storey-height panels, supporting external brick cladding.

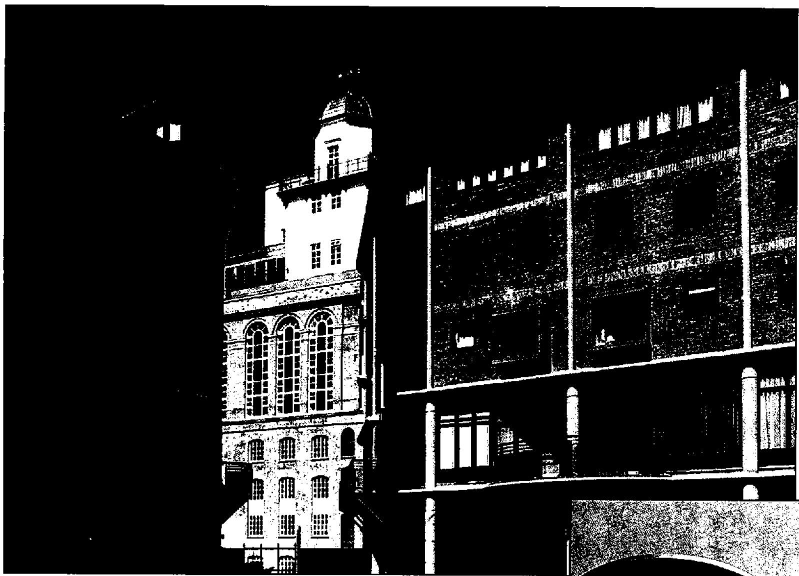


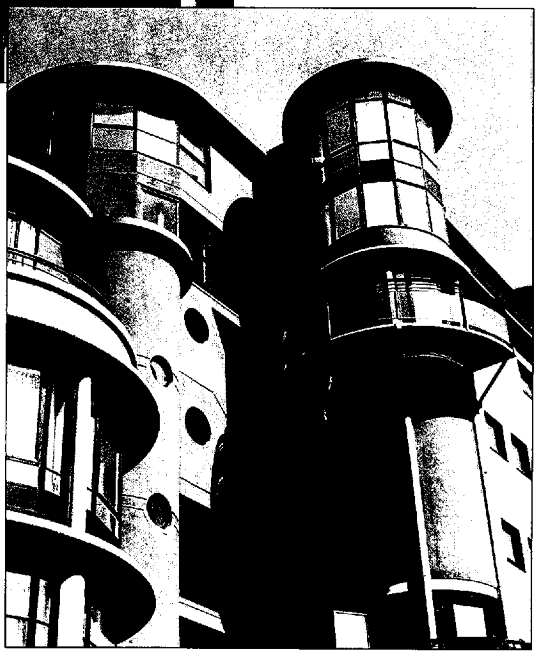


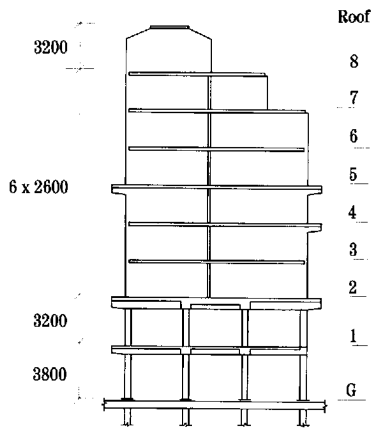
Typical floor layout

■ HORSELY DOWN SQUARE, LONDON

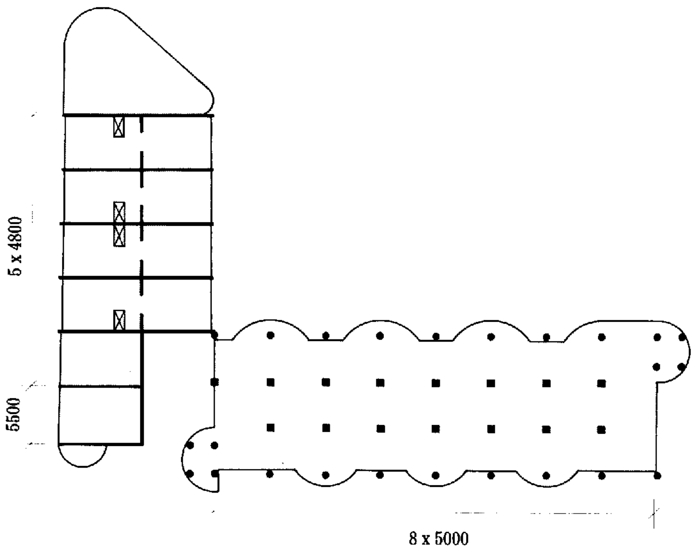
- Varied plan form with crosswall and framed construction.
- Curved cantilevered edge units.
- External high quality visual concrete.



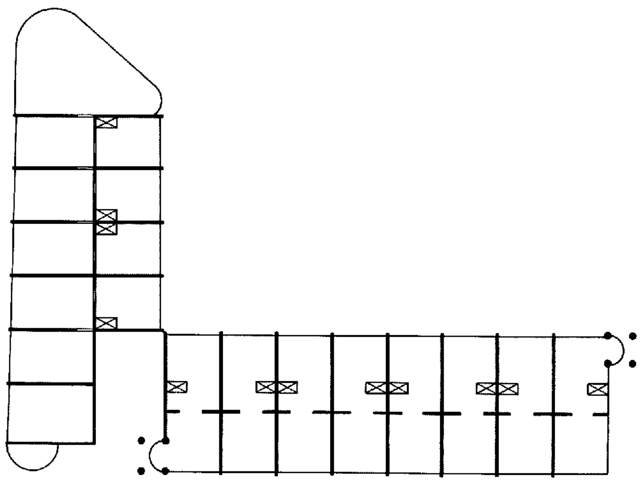




Typical section

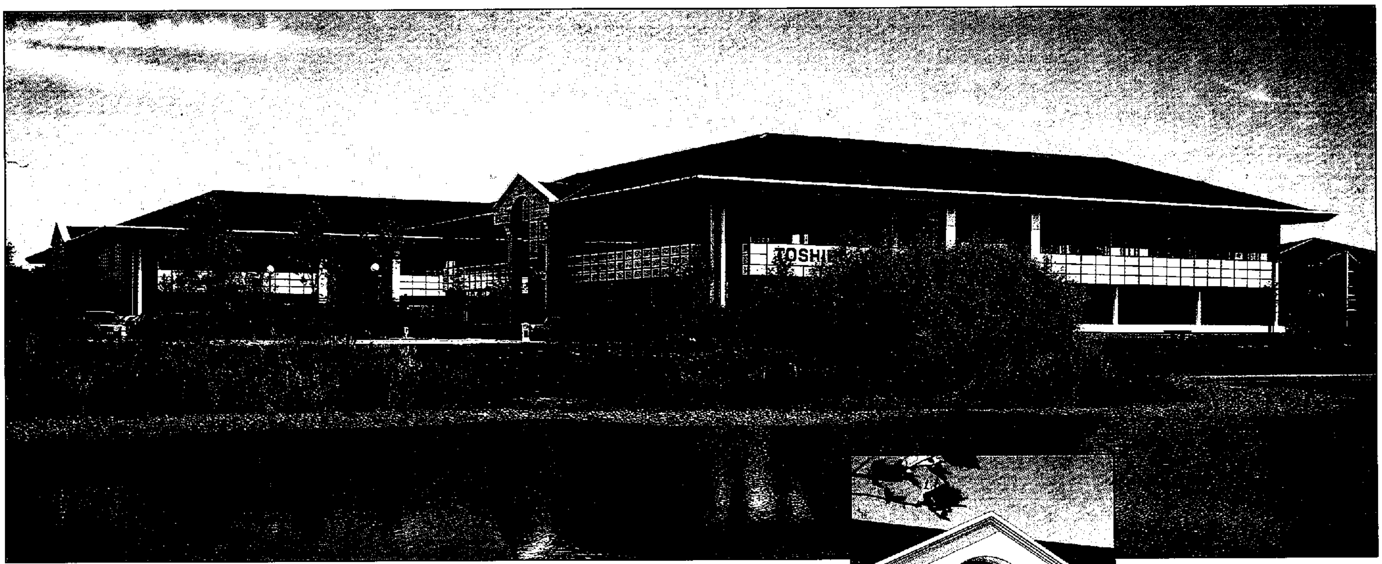


Blocks B & C – layout of floor two

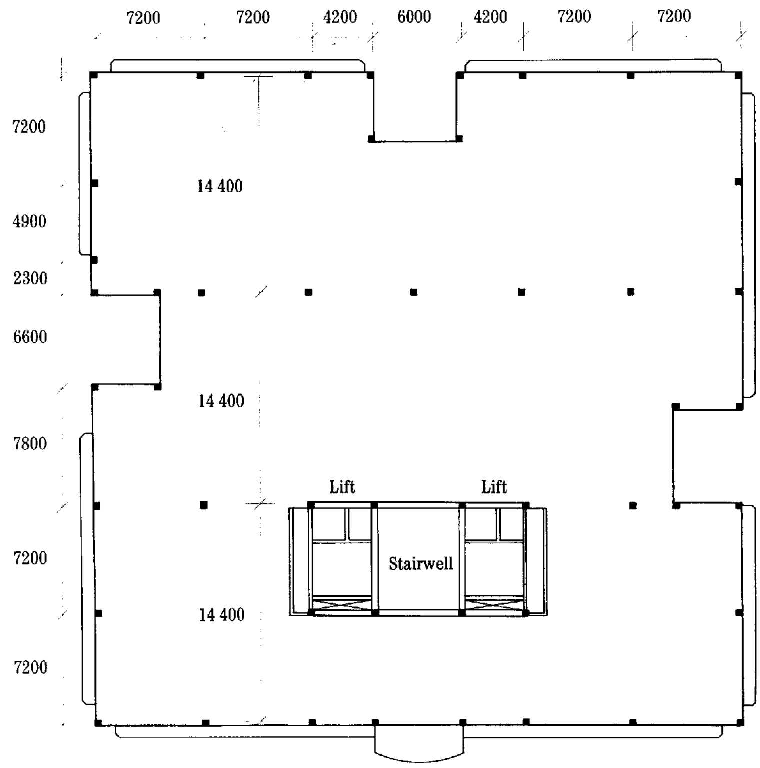


Blocks B & C – layout of floor three

■ ■ ■ WATCHMOOR PARK, CAMBERLEY

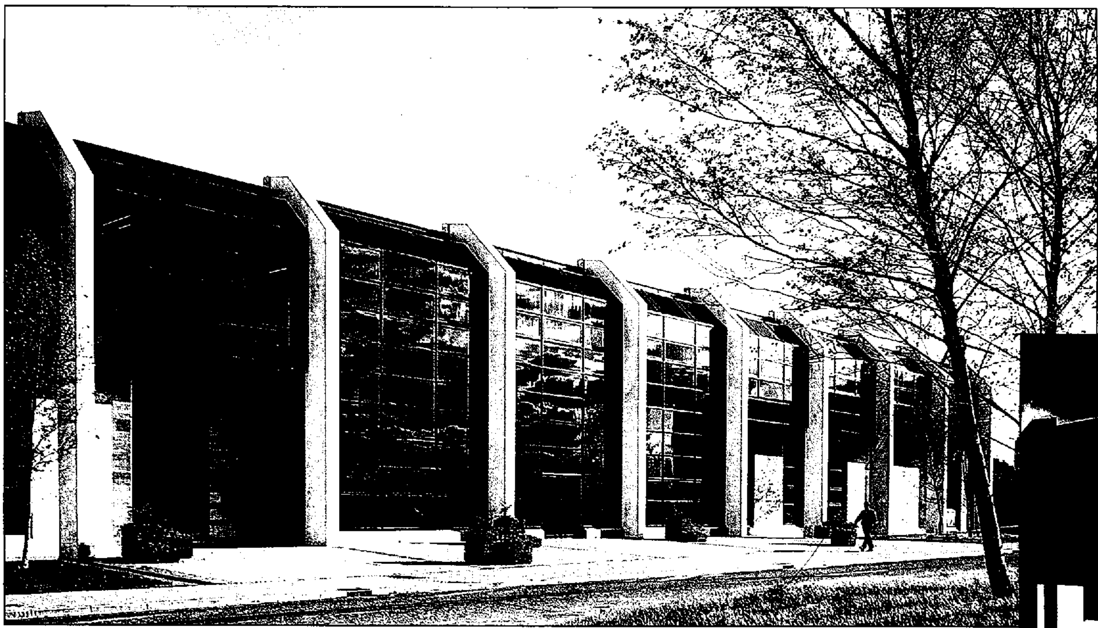


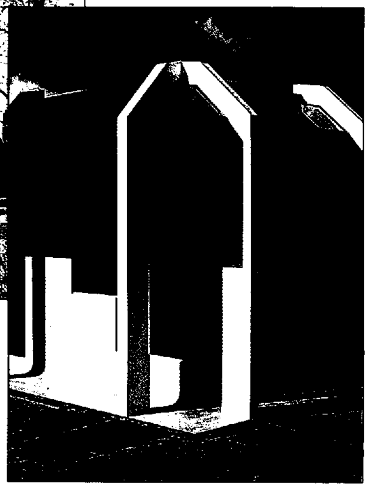
- High-tech precast composite frame.
- Long-span floors with cantilever edge strip.
- Precast ground beams.



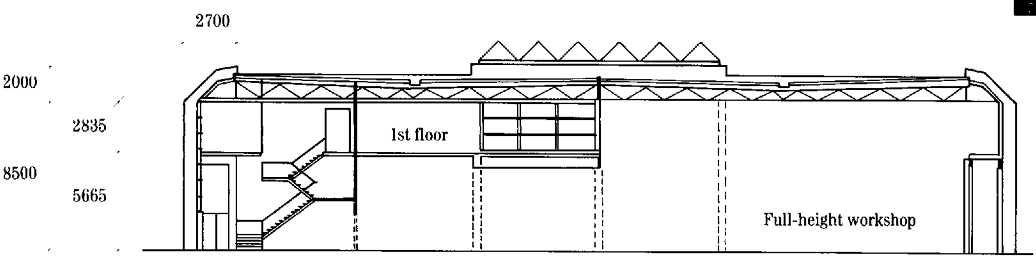
Typical floor layout

■ SUNBURY BUSINESS CENTRE

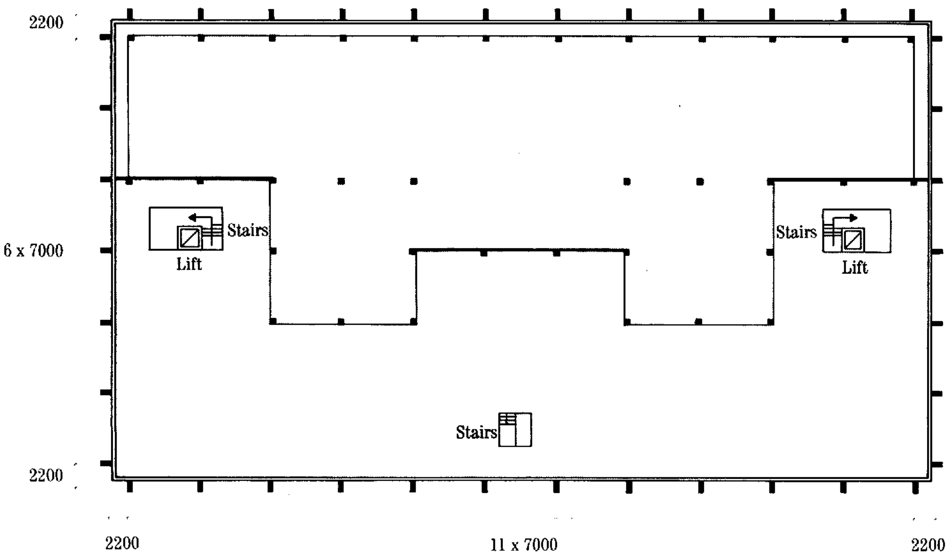




- Precast composite frame with angled two-storey, one-piece columns.
- High quality visual concrete to external columns.



Typical cross section



First floor layout

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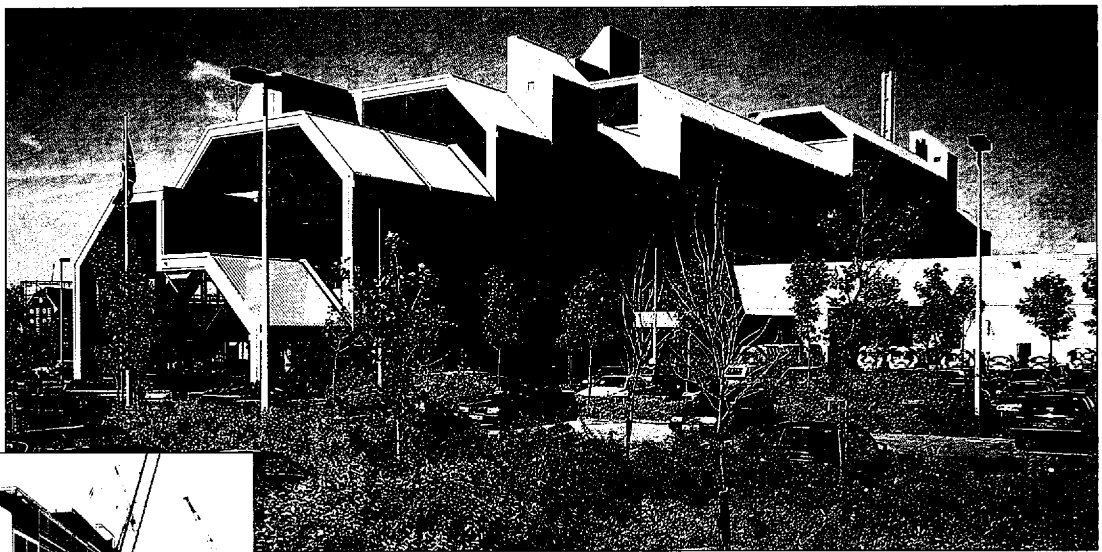
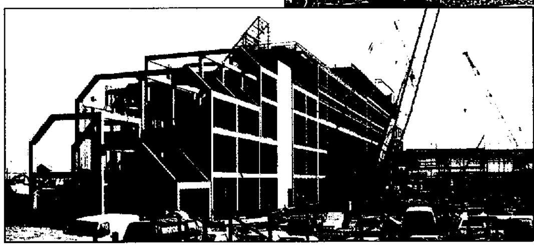
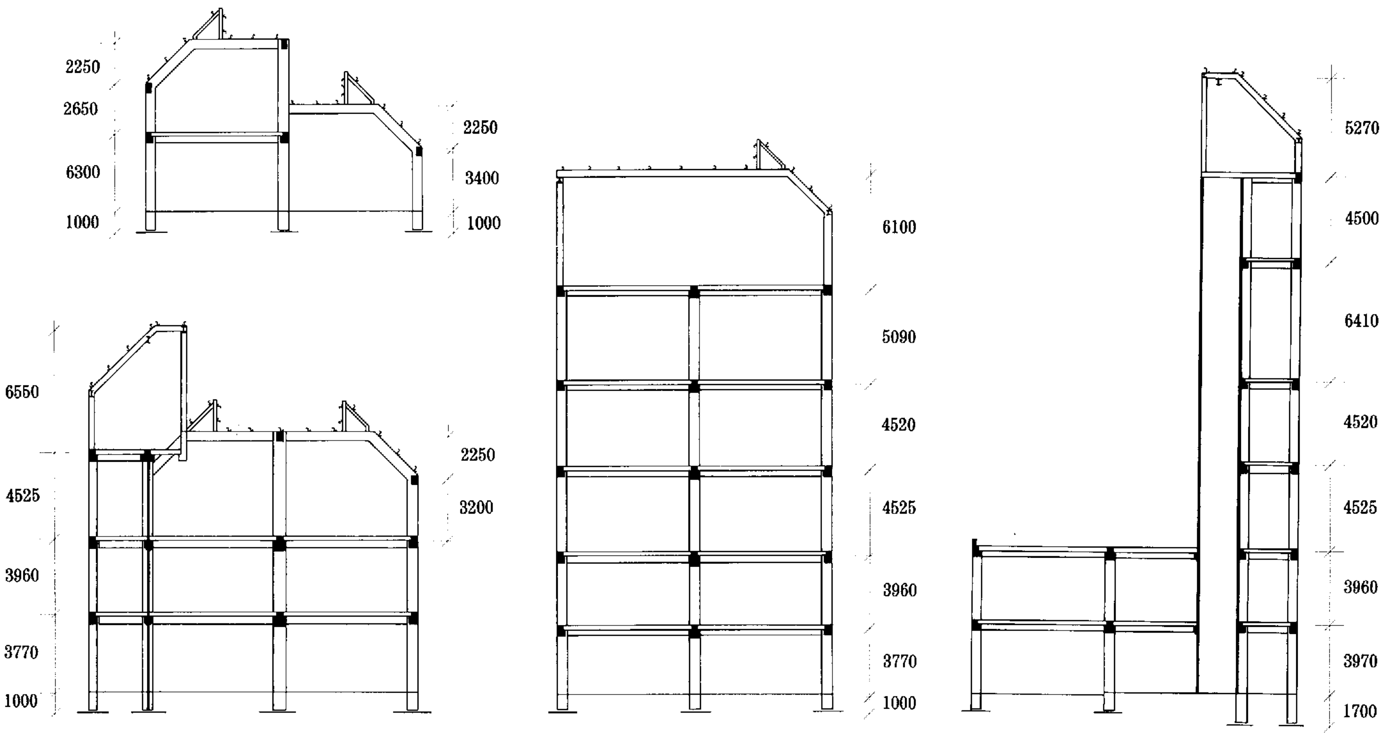
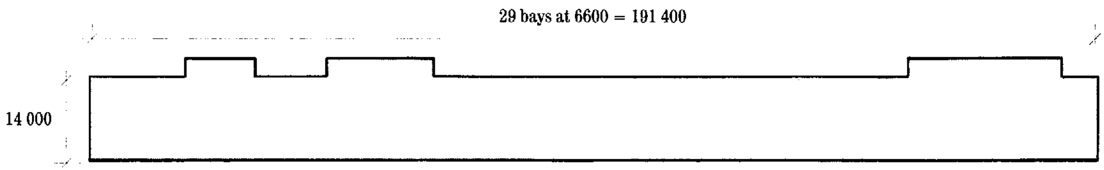
■ DAILY MAIL, SURREY DOCKS, LONDON

- Large, complex precast frame.
- Varying building cross-section and floor height.
- Cranked concrete and steel roof beams.







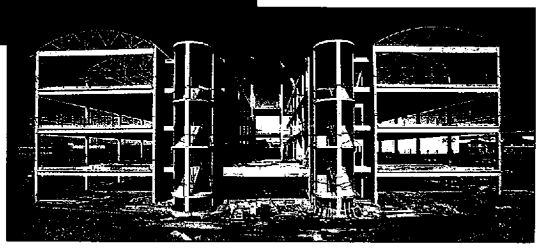


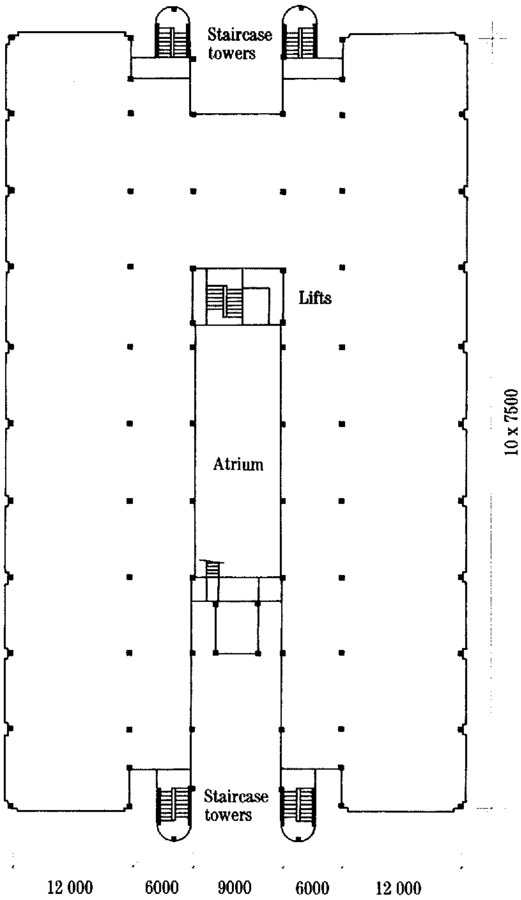
General building plan

51

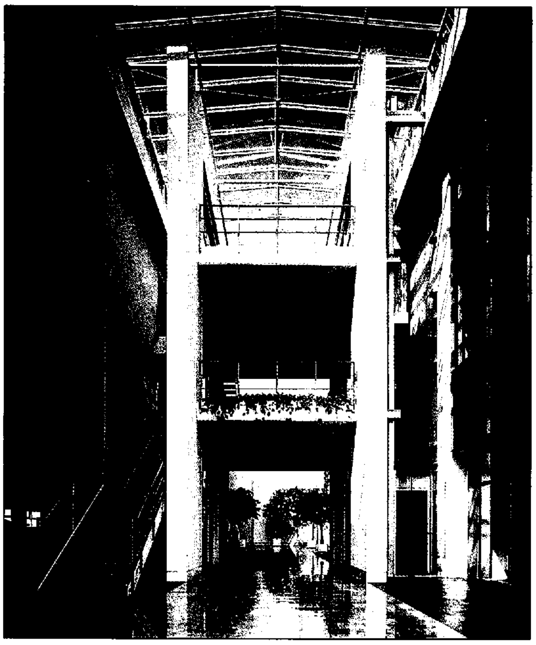
■ IMPERIUM, READING

- Precast composite frame.
- Long-span floors.
- Curved staircase landing units.





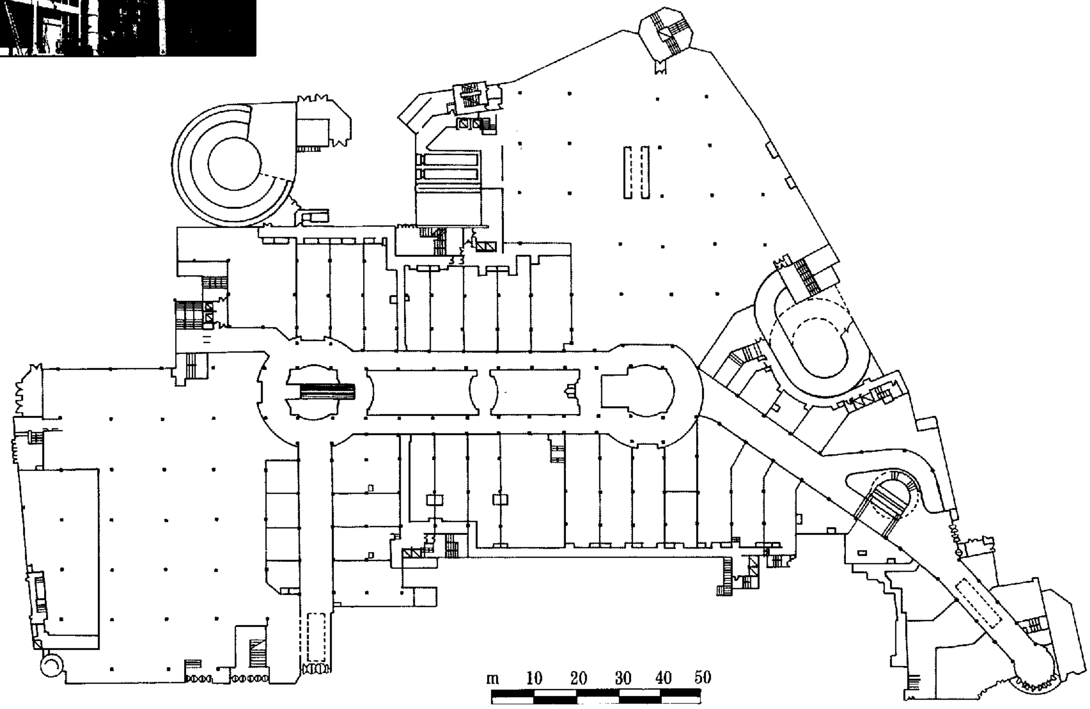
Layout of floor one



51

THE SHIRES, LEICESTER

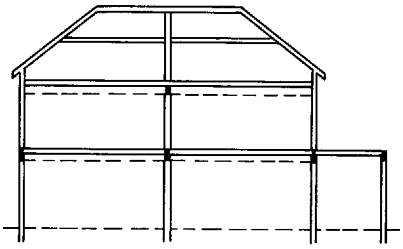
- The largest precast concrete retail development.
- Variability in plan form and structure.
- High quality visual concrete feature columns and other units.



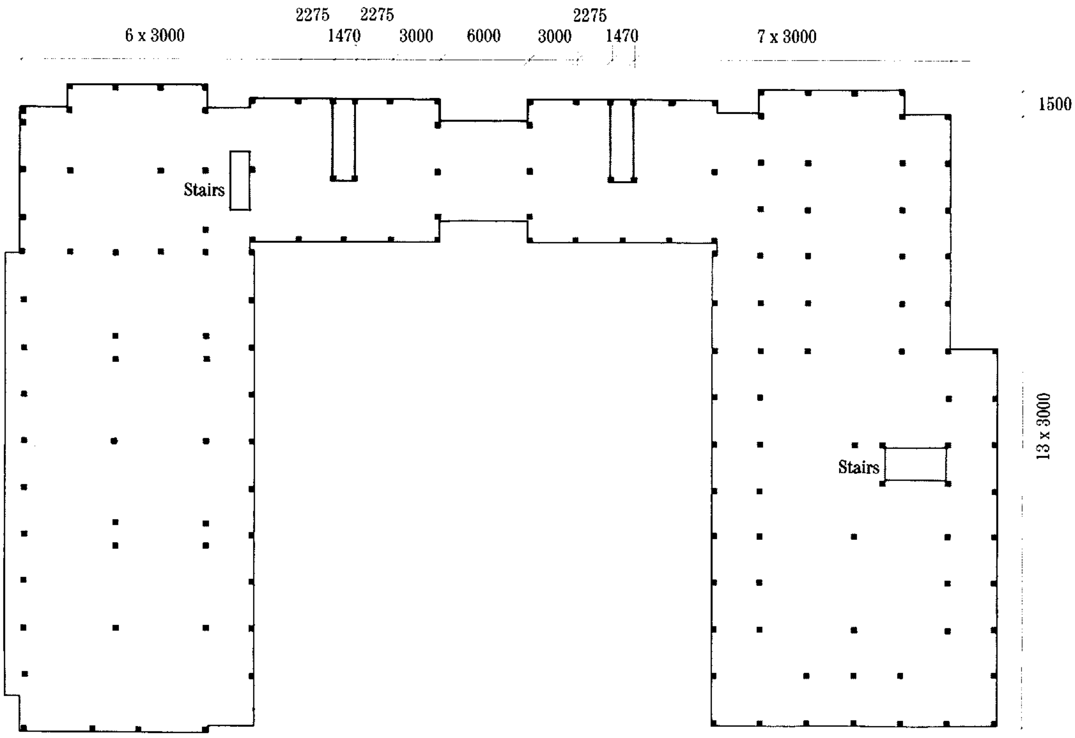
General layout – upper ground floor

WATERMEAD, AYLESBURY

- Frame with varying column grid layout.
- Expressed columns and curved beams.
- Cantilever floor slabs.



Typical cross section

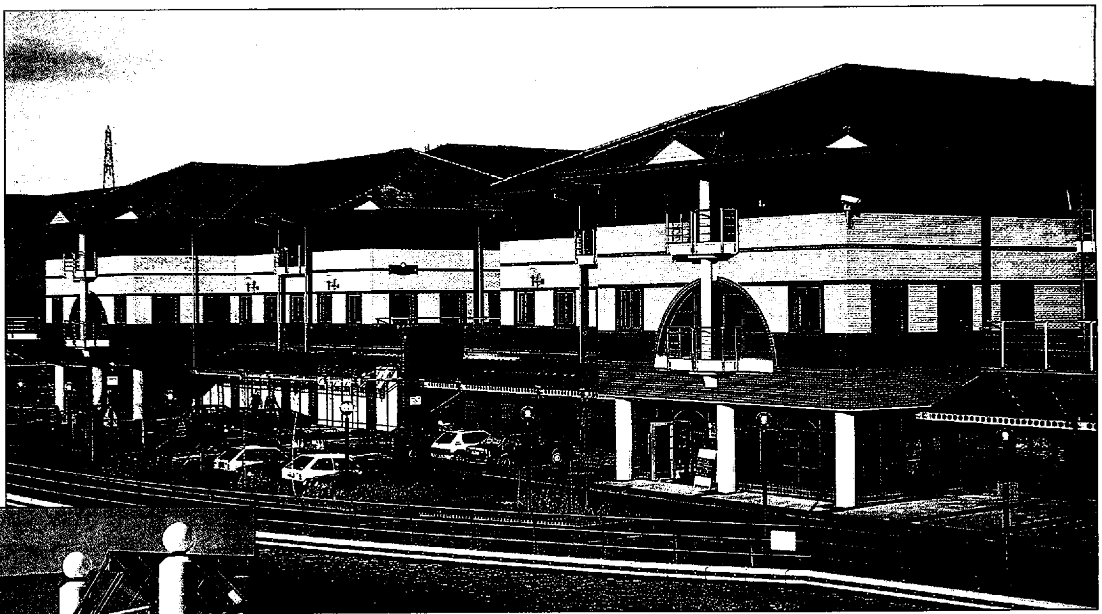


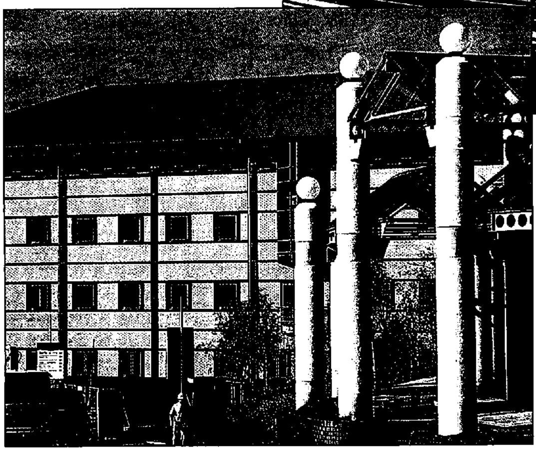
Typical floor layout

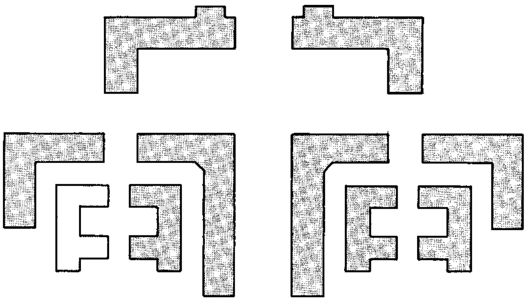
11

■ THE WATERFRONT, BRIERLEY HILL, DUDLEY

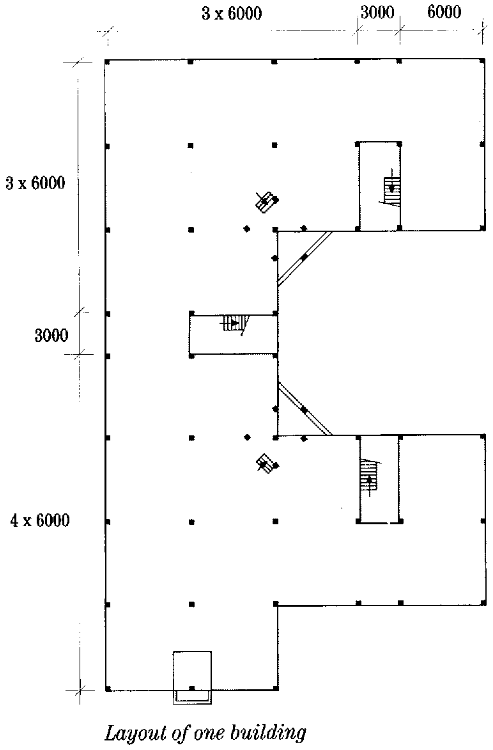
- Large commercial development.
- Ten structures of varying height and plan form, some interlinked.
- External feature columns







General building plan



Layout of one building

2

PROCUREMENT

■ ■ APPROACHES TO PROCUREMENT

There are a number of routes to the procurement of a precast concrete frame, each tailored to meet the requirements of the job, the client, the contractor and the design team. The procedures may be adapted to take account of cash flow, budget, programme, design sequence, materials, construction methods, and many other factors. The traditional route of design by consultant, followed by competitive tender and building by contractor, is nowadays frequently modified or abandoned in favour of methods such as design and build, turn-key, project management, negotiated contract, target price, or cost plus. Whichever route is chosen, a precast concrete structure imposes its own discipline on the procurement procedure.

The traditional procurement route is probably the best for in-situ reinforced concrete structures. Here the design and detailing are within the control of the consultant and the work can be fully described, specified, measured and competitively tendered for. The contractor is then totally responsible for the construction and is not normally required to contribute to the design.

However, the traditional procurement route often has to be adjusted to deal with specialist design and factory production, both of which will generally apply to precast concrete.

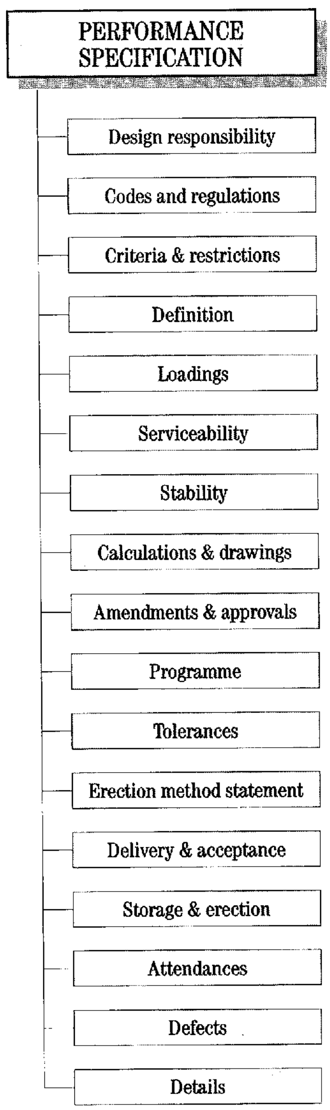
Where a supplier or contractor has developed a special product or technique, the benefits of this expertise should be passed on to the client. It is not appropriate for the consultant to undertake, or to be responsible for, these aspects of the design, but he must be satisfied that the specialist design has been properly executed and must integrate it into the overall design of the project. To this end the consultant must be satisfied with the competence of the specialist designer, set out the design requirements and ensure the proper flow of information between specialist and consultant.

The consultant must also ensure from the outset that both factory and site appreciate each other's aims and problems. Their demands on each other and on the design team must be defined in advance and closely monitored throughout the project.

Most engineers will be familiar with concrete as a structural material and their previous knowledge of in-situ concrete will still apply. The design and detailing rules for precast concrete are generally the same as those for in-situ concrete, and concrete mixes, cover to reinforcement, compaction and curing all remain critical in achieving a high quality product. The difference is that precast components are produced off-site, in a factory. Precast concrete frames allow the engineer to combine the benefits of this structural material with the advantages of sophisticated factory production techniques.

By considering the stages in the procurement of a precast concrete frame in more detail, the remainder of this Section will act as a guide to the engineer, architect or client who is unfamiliar with the process. The engineer will also need to address the detailed design aspects in order to develop a performance specification for the project (Figure 2.1). Example specification clauses are given in the Appendix, and can be adapted for use as a working document. Used together, this guidance and specification should ensure that correct actions are taken at the right time, common problems and pitfalls are avoided and a successful precast concrete frame building is procured.

■ 2:1 Design aspects for performance specification



SUITABILITY FOR PRECAST CONSTRUCTION

The first requirement is to identify whether the building is suitable for construction in precast concrete. There are no building materials or methods which are applicable to all, or even to the majority of, situations. It will always be a matter of selecting the material and method best suited to the particular circumstances.

This Section is primarily concerned with the procurement of what are often mistakenly referred to as frame 'systems'. The frames described in this publication are hardly systems, even though there will be considerable repetition of components. Features of individual systems generally relate to their particular column-to-beam connections. Precast concrete frames may be more correctly considered as methods of construction using precast concrete components and offering notable flexibility of design.

It is commonly thought that precast concrete construction demands long lead-in times to allow for design, procurement, mould manufacture, and production and supply of the components. A precast concrete frame requires the precaster to become involved early in the design process. If the project and contractual arrangements permit this, the lead-in time is seldom a problem, and the frame can usually be supplied by the time the foundations have been constructed; production and erection can usually overlap.

Those structures best suited to factory pre-casting will exhibit a degree of repetition in their structural grid, spans, loadings, storey heights, member sizes, etc. Such repetition is not always obvious. Sometimes it may be imposed and sometimes it may be a rationalisation of initial confusion. But the search for common elements and features, i.e., repetition, should not be confined to precast concrete construction; it is a worthwhile exercise in any design.

Buildings with an orthogonal plan are ideal for precasting, because they can be framed simply by beams and columns, but it is possible to design frames for irregular plans, as illustrated in Sections 1 and 3, and all will benefit from a degree of repetition. The ideal building is likely to be from two up to a maximum of about 15 storeys high. When the height exceeds this, column sizes may become a problem and resistance to wind forces may demand frame action or bracing incompatible with normal precast concrete frames (see Section 6).

Within this height range, readily achievable concrete strengths of 50 N/mm^2 and more will enable sensible column sections to be used. The plan form should be such that lateral load resistance of the pin-jointed frame can be provided by the horizontal diaphragm action of the grouted or screeded floor slabs, and by bracing or shear walls at stairs, lifts and elsewhere (see Section 6).

Floors with spans up to 9 or 10 m, and with normal loadings and depths, may be designed using hollow-core floor units. Greater spans of up to 14 or 15 m are possible using extra-deep hollow-core units, and up to 22 or 23 m with deeper double-tee beams (see Sections 4 and 6 for further details).

Storey height restrictions and provisions for services will determine the acceptability of the beam depths required and hence the suitability of precast concrete.

CONFIRMING PRECASTABILITY

If the engineer considers that precast concrete may be suitable, a second opinion may be obtained by seeking the advice of the precasting industry. The engineer can do this by a direct approach to a precaster, or by contacting the Precast Concrete Frame Association, from where the enquiry will be passed on to suitable member companies. The frame manufacturer will quickly tell the engineer whether or not precasting will be appropriate for the particular project, either as designed or with modification. The manufacturer will also be able to give an indication of the programme and budget.

The engineer should then be ready to suggest that the client and design team look into the precast concrete frame in more detail. A function analysis may be carried out to compare precast concrete, in-situ concrete and steel frame buildings. The results of this analysis may be assessed conveniently by using a table.

It should be possible to obtain prices for budgeting purposes from one or two precasters, based on outline drawings and a short list of performance criteria. At the same time the engineer can produce a rudimentary steel frame design and obtain cost indications for this from one or two steel fabricators. A parallel exercise in in-situ concrete can be priced with the assistance of either a quantity surveyor or a contractor's estimator. It is not difficult to obtain budget prices from a reliable contractor.

Precasters vary in the form of budget price which they are prepared to give. It may be a simple approximate cost per square metre, a lump sum, or a detailed breakdown. Many precasters have computerized costing systems which enable them to price a job quickly, provided that the engineer gives them sufficient information.

The budget prices obtained in this way should be included in a comprehensive comparison of the advantages and disadvantages of the alternative construction methods for the project.

There are a number of matters which the comparison should consider.

- How well does the construction method respond to the architect's requirements? Is the design concept unduly compromised by the dictates of the structural material?
- Which method and material best satisfy the

services engineer's demands for horizontal and vertical distribution zones, penetrations, thermal mass, acoustics, etc?

- How adaptable is the structure required to be? Will it tolerate late development of the design? What alterations to the building form and function will the structure permit in its later life?
- What are the particular procurement requirements for the alternative forms of construction? What are their lead-in, procurement, delivery and erection times? Which of these are critical for the project?
- What is the cost of the structure and how do the alternative structural solutions affect the overall cost of the project?

The comparison must be on a genuine like-for-like basis. The precast concrete frame will generally include floors, stairs, walls and fire protection. It is important to ensure that these are included in the steel frame appraisal. Foundation costs will also vary and should be taken into account. Precast concrete will include a greater proportion of the construction work in one subcontract and may permit earlier access to following trades. Fast-track procurement and construction may offset a higher capital cost by reducing financing costs and securing earlier rental income.

The analysis must be detailed and thorough, and can only be carried out effectively by somebody with a sound appreciation of each of the alternative forms of construction. The engineer plays a vital part in this analysis and ensures that vested interests and biased opinions do not leave the client with a second-best design.

THE CHOICE OF CONTRACTUAL ROUTE

If the comparison leads to a decision to build a precast concrete frame building, the most appropriate contractual route must then be chosen. The degree of involvement by the client, architect, engineer, etc., may vary widely, depending on the type of precast concrete structure being designed.

At one extreme, where the building is simple and the design criteria are briefly stated and readily checked, a warehouse or car park for example, their involvement will be minimal and the engineer may play a minor role. In this case the 'contractor design-and-build' route may be satisfactory.

At the other extreme is the complex or demanding structure where precast concrete has been selected as a direct result of that complexity, or the building's special requirements for accuracy or finish. Under these circumstances the architect and engineer may undertake all the design and detailing, with the precaster or main contractor acting as adviser and producer, but having no responsibility for design. In this case the consultant and contractor will play their traditional roles.

Between these extremes lies the range of everyday building projects, where a specialist precaster's framing method is likely to provide the most cost-effective structure. To this end specialist precasters must not only be permitted, but actively encouraged, to contribute to the design and to become members of the design team from the date of their appointment. The contractual procedures must be designed to achieve this.

Time is generally very important in building procurement. Often it will be necessary to start the precast concrete design before appointing the main contractor, or even before preparing the main contract tender drawings. The architect and the engineer generally cannot progress, much less complete, the overall design of the building without the precaster's design input.

Positions, sizes and details of structural members will have to be known so that the exact layout can be finally decided. The services engineer will need to know beam locations and depths for planning duct work, and the structural engineer must know the column loads before designing foundations. This should not be a one-way process in which the architect passes preliminary layouts to the precaster for frame design, then these are sent back for detailed design by the architect and others. It must be an interactive dialogue, with all parties progressing the design together.

This interaction is frequently achieved by nominating the precast concrete specialist contractor following either a formal competitive tender for the design, supply and erection of the frame, or negotiation with a preferred precaster. The latter may be necessary if the building design calls for a specific manufacturer's framing method but, as mentioned before, systems have more similarities than differences, so the need to restrict the choice of manufacturer should seldom arise.

Normally, therefore, tenders for the frame will be followed by some form of direct contract between client and precaster until the main contractor is appointed and is instructed to take on the precaster as a nominated subcontractor. The JCT '80 form with subcontractor design element is generally appropriate and should be recommended to the client. Avoidance of nomination and attempting to persuade the main contractor to take on the precaster as a domestic subcontractor, should be avoided because it is impracticable, and tends to distance the design team from the precaster.

SELECTING AND SHORT-LISTING THE PRECASTER

When a precast concrete frame has been decided upon, selecting and short-listing precasters is the next and important step. The engineer's advice on this selection and short-listing should be based on very thorough research. The tender submissions will need to be assessed (see *Evaluation of tenders and appointment of precaster* on page 29), but careful selection of the tenderers to be invited will result in the lowest tender being more readily acceptable. The engineer's preliminary enquiries will have produced a list of names of those precasters interested in, and capable of, the type of frame work required.

When assessing precasters for the first time, the engineer is advised to start by drawing up a list of about six, and asking these to submit details of their company, including brochures, lists of current and recent jobs and any other information which is considered relevant or of interest. They should be asked for the names of a few architects, engineers and main contractors for whom they have worked, and these should be contacted for their opinions. When this desk study has been completed, the engineer should visit the precasters' factories and design offices, in reverse order of preference. In this way the engineer's expertise will increase and the most discerning judgement will be applied to those most likely to be included on the tender list. This can prove an exhausting exercise in both logistics and intelligence gathering.

It is advisable for a questionnaire to be drawn up to ensure that the questions asked and the points explored are the same with each precaster. The criteria for selecting a precaster are illustrated in Figure 2.2 opposite. Areas of enquiry are listed below.

- Company profile – ownership, history and fields of activity, turnover and the proportion of this which is related to precast concrete frames.
- Manpower – design, production, administration and site.
- Interest in the project – capability, capacity and experience.
- Design resources – in-house or external; examples of work for previous projects.
- The frame – features and history of development.
- Factory or factories – locations and output; general conditions – heat, light, power, drainage; mould making, reinforcement, concrete production, casting and post-casting.
- Quality control – theory and practice; inspection of current production and stock; manuals and records.
- Erection arrangements.
- Sub-letting and buying in.

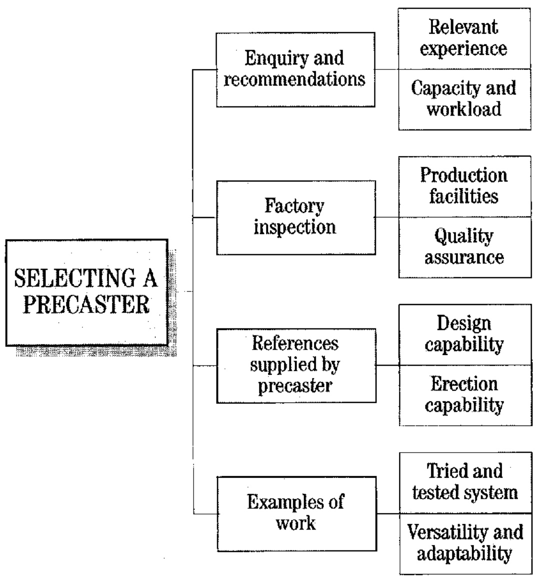
During evaluation it is essential to look for a precaster with experience relevant to the proposed construction. Where a structural frame needs to be fully designed and supplied, the precaster must be highly experienced and have a good track record in structural frames, preferably supplying a tried and tested structural frame method with standard component and connection designs, and with test data to support it.

Although factory-produced units are of a high standard, particular attention should be paid to the design, detailing and construction of the structural joints. The engineer should ensure that the precaster can provide comprehensive information on the various forms of connection.

The engineer should seek a frame method with sufficient versatility and adaptability to satisfy the requirements of the building design without needing too much in-situ concrete, or too many special steel components. However, it can often be efficient to use steelwork, in conjunction with a precast concrete frame, for bracing or to form the roof structure and plant room. Such composite frames are discussed further in Section 3.

The engineer may also seek a precaster capable of undertaking the full design. The precaster's own in-house designers are most likely to understand the frame method and fully appreciate its potentials and problems. Most of the better known precasters have such designers, and many have sophisticated computer-aided design and drafting systems. The engineer should be wary of a precaster who engages the design services of an independent consultant, unless there is evidence that the consultant has had a long-standing relationship with the precaster and fully understands the frame method on offer.

2:2 Criteria for selecting a precaster



When making an initial factory inspection it is most important to check that the main production workshops are under cover, sufficiently spacious, well-maintained, warm, and well-lit. The quality control procedures for production should be inspected, including mould manufacture and maintenance, fabrication of reinforcement cages, compaction and curing of concrete, lifting, storage and indentification of units.

The quality control manuals should be readily available and the engineer should check that records are maintained. The laboratory should at least be able to test aggregates and concrete cubes.

Test machine calibration records should also be checked. Many of these checks will be considerably simplified where a third-party quality assurance scheme is in operation at the precast concrete factory (see Section 5).

The erection of a precast concrete building is a skilled operation and should not be entrusted to an inexperienced contractor. Some precasters employ their own erection teams and some use a specialist subcontractor, in which case a check should be made to ensure that the subcontractor has sufficient experience of erecting that precaster's frames.

INVITATION TO TENDER

Following these inspections and discussions with a number of precasters, and having studied their budget price proposals, the engineer should be able to prepare drawings and a specification, and invite competitive tenders.

One approach for the engineer is simply to issue the architect's preliminary floor plans, sections and elevations, and ask the tenderers to propose a suitable structure. It might be argued that this provides the precaster with the maximum design flexibility and ensures that the client gets the full benefit of the precaster's expertise and creativity. This may be true, but it makes comparison of the tenders very difficult because design criteria and constraints may not be fully stated or appreciated and they may be interpreted in different ways. It may be necessary to ask for amended tenders when these various proposals have been examined.

A more satisfactory approach is for the engineer to prepare indicative frame drawings, to be read in conjunction with the architect's plans, and showing the following.

- The basic layout of the building.
- Notional beam and column positions and structural zones.
- Limitations on beam and column sizes.
- Locations of stairs and lifts.
- Constraints imposed by the services – particularly ductwork, and major voids in floor slabs.
- General and specific loadings.
- Foundation, roof and cladding principles.

The drawings should be accompanied by a performance specification; this should not be too restrictive, but simply specifying the materials and workmanship will not be adequate. What is required are definitions of the design criteria and the performance of the frame in service. This form of specification may be sufficient if the architect and engineer will not be involved further in the design, such as when the tender is for the design and construction, but it is more likely that the engineer's duties will include ensuring the correct design, production and erection of the precast concrete frame, and integrating it into the overall design of the building. This will require a more broadly based specification, (see Figure 2.1), not only defining performance, but also establishing the ground rules for design, production and erection.

The current National Building Specification (NBS) does not cover precast concrete frames. Some sections of the NBS cover frames of other materials, but not generally those with any significant element of design by the contractor. However, there is a section on precast concrete cladding, which has been used as the basis for the performance-related specification for precast concrete frames in the Appendix. An adapted version of this example specification has been used effectively on a major precast concrete frame project and it is hoped that, by using it as a model, a more consistent approach to frame procurement will be obtained.

The drawings and specification will constitute the main part of the invitations to tender and these will be combined with instructions to tenderers and details of information to be provided with the tender.

The appropriate tender period will relate to the size and complexity of the job, but if the tenderers had been approached for budget prices they will have prior knowledge of the job and a tender period of four weeks would seem sufficient. A period of only three weeks may be possible, and anything more than five weeks is unlikely to be productive.

EVALUATION OF TENDERS AND APPOINTMENT OF PRECASTER

When drawing up the invitation to tender, the engineer will have attempted to ensure that all precasters' tender proposals will be reasonably comparable, but differences must be expected and identified. Where errors in interpreting the tender documents have occurred, corrections should be permitted, but confidentiality must be respected and no attempt should be made to distil the best design aspects from each tender. The precasters' proposals will reflect their own particular preferences and optimum designs.

When tenders have been received, the client may press the engineer to accept the lowest. It is often argued that the time to avoid unsatisfactory tenders or tenderers is when drawing up the short-list, not when selecting the successful tender. There is no satisfactory response to the client's observation that the contractors were vetted before being invited to tender, so none can subsequently be considered incompetent. However, the tenders may vary in detail or contain qualifications, so the engineer should be wary of recommending acceptance of the lowest without a detailed scrutiny.

It is not the frame cost alone that matters, but its effect on the overall building cost. The overall cost may be affected by the earliest dates for commencing and completing the frame erection, the sequence of handovers to following trades, the adaptability of the frame to suit late amendments to the design, its ability to provide an early dry enclosure, its interaction with other building elements, the incorporation of loadbearing cladding and many other considerations.

The essential information needed by the engineer to make an assessment of the tenders will be:

- Description and extent of the frame.
- The price.
- The method and sequence of erection.
- The programme for design, manufacture and erection.

Further details of these items are given in the example specification in the Appendix.

It may be unreasonable to expect all tenderers to provide such detail. An alternative approach is to ask all tenderers to submit their total price, the time required from ordering to commencing erection and the time to complete erection. At the same time the tenderers should be informed that they may be required to submit such documents as a detailed breakdown of cost, an erection method statement, a bar-chart programme, and an information release schedule within, say, seven days, if their tender is of interest to the client. In this way the engineer ensures that there is access to all the data required for evaluating the tender, without involving unsuccessful tenderers in unnecessary work.

The engineer will prepare, or assist the quantity surveyor to prepare, a tender report and recommendation for the client. Before the report is completed, it is advisable to hold a non-committal pre-contract meeting with the preferred precaster, or perhaps with the two front runners. At this meeting, or these meetings, any remaining queries should be answered and every attempt made to eliminate misunderstanding. There can be no justification for avoiding difficult issues at this stage. It is in the interests of all parties to get the right price for the right job. Any special conditions attached to the tenderer's offer must be either formally agreed or withdrawn. The tender must be amended to incorporate any supplementary points agreed in the post-tender correspondence or at the pre-contract meeting. Alternatively, such letters and meeting minutes should be written into the contract.

Acting upon the tender report, the client either authorizes the architect to nominate the precast concrete contractor and instruct the main contractor to enter into a sub-contract with the precaster or, in the more likely event of this taking place before the main contractor is appointed, the client issues a letter of intent to the precaster. The letter of intent is both a notice of intention to nominate the precaster in due course, and an interim instruction to proceed with the work, or part of it. For example, the design work could start, and possibly also the manufacture of moulds and ordering of materials; even some of the production could commence before nomination, in exchange for direct payment by the client as agreed.

When work is in progress on the basis of a letter of intent, it is important for all variations to be recorded so that, when the time comes for nomination, the main contractor can be advised of the true nature and content of the precast concrete subcontract works. In this intervening period the architect, engineer and quantity surveyor must act on behalf of, and have regard to the interests of, the prospective main contractor.

SUB-LETTING BY TENDERER

Tenderers must be required to identify any work that they propose to sub-let or buy in. Sub-letting and buying in are very common in precast concrete frame work and should not be rejected out of hand. The circumstances and specific proposals must be considered. Because a precaster has a particular capability it does not necessarily mean that it will be used. In some circumstances it may be more economic to sub-let or buy in that capability.

Most precasters have their own in-house designers and site erectors, but some have long-standing arrangements with an independent consultant who does their design, and some may choose to employ the services of a specialist erector. The precaster may propose to sub-let the production of all or some of the frame units. The principle to be followed with any sub-letting, however, is that the appointed precast concrete company must retain overall responsibility for the work for which it is to be paid, know what is going on, and always be in control.

The same applies to bought-in items, such as cast-in fixtures and fittings, prefabricated reinforcement cages, moulds, hollow-core floor units and double tees. The appointed precast concrete company must satisfy itself and the engineer that the bought-in items can be procured on time and to specification, and these must be continuously monitored.

DESIGN

Before an order or letter of intent is issued, two key matters need to be addressed.

Firstly, the precaster's key personnel need to be identified. The person in overall charge of the project must be named, but by the nature of the precasting industry this may not be a simple matter. There may be many departments involved – sales, estimating, design, contracts, production and erection. The job will pass from one department to another and from one manager to another, perhaps with a director maintaining a watching brief. Nevertheless, there must be a project manager responsible for co-ordinating the activities of all departments and to whom all problems which arise are ultimately referred. Day-to-day contact may be with the manager of a particular department, but to ensure control of the project at all times, all communications should either be through, or copied to, the precaster's designated project manager.

Secondly, a detailed information release schedule must be agreed. It would be ideal for the precaster if the architect's design were frozen on the day the order is placed, but ideal for the architect if the opportunity to amend the design remained open until the building was on site. These are two unrealistic attitudes, neither of which is in the best interests of either party. The precaster's design will identify areas where architectural change would be beneficial to the precaster and if the architect's options remain open too long such undisciplined design could be costly and chaotic. Therefore, all parties must try to understand each other's design processes and agree priorities for their decision making.

When preparing the tender, the precaster will have carried out a preliminary structural design and will have determined the outline of the structural frame, together with the sections of the beams and columns, and the locations of the walls required for stability. The designer of the precast frame must try to anticipate the requirements of the architect and the services engineer. As early as possible the architect and engineer should either confirm their acceptance of this outline design, or agree any amendments with the precaster, including, for example, positions of columns, shear walls and service holes. The aim is to establish a workable structure and to stick to it, thus enabling the detailed design of the frame to proceed.

If flexibility is to be retained it must be designed in. During this early development of the design, the precaster will be preparing the general arrangement drawings and calculating the loads on members. Early in the programme the engineer will need to know the vertical loads on columns and walls and the stability forces required to resist wind, so that the foundations can be designed. If piled foundations are proposed, preliminary column and wall loads may be needed very quickly so that the piling contractors can design and tender. It may be necessary for the engineer to make an assessment of these loads if more precise information is not available.

When the general arrangement drawings and supporting calculations are complete and the architect and engineer have no further comments to make on them, the detailed design and drawings of individual components can commence.

The preparation of these drawings is a major undertaking for the precaster's drawing office, which is not often appreciated by the architect and engineer. They may be tempted to look again at the general arrangement drawings and to make 'minor' changes. This temptation should be strenuously resisted, because the general arrangement drawings are the fixed information base for the precaster's production, including the procurement of moulds and ordering of materials. For the sake of budget and programme and harmonious relations with the precaster, it is advisable for an approved set of general arrangement drawings to be looked upon as equivalent to an erected frame even though no units may have been cast. This is the difference between the highly organized factory production of precast concrete, and in-situ concrete construction, where the formwork can be adjusted up to the moment the concrete is placed.

The precaster must also prepare a set of floor plans which identifies each individual floor unit. These plans will show where each unit is to be placed, and will schedule each in terms of strength, length, width, holes and inserts, if any. Both the architect and services engineer must be ready to define their requirements for all holes in the floors in time for the precaster to prepare these drawings. The sizes and positions of holes affect the design of the slab, and the configuration of the prestressing strand has to be adjusted to suit the sizes and positions of holes. The ability to drill on site is limited by the position of the prestressing strands which occur in the ribs of hollow-core floor units (Figure 4.12), and double-tees (Figure 4.13). In precast solid plank floors (Figure 4.14), the strands may be at about 100 mm centres.

In screeded slabs, robustness is provided by mesh or bar reinforcement in the topping, and this also provides or contributes to the diaphragm action of the floors. In unscreeded slabs, concreted joints between precast units, and reinforcement incorporated between floors, beams and columns, will provide tying and ensure diaphragm action (see Section 6). Further drawings will be required to detail any screeds or other in-situ work.

The precast designer will be required to provide full structural calculations, both for the engineer and for submission for Building Regulations approval.

All these activities must be identified in the information release schedule. All parties involved in, or affected by, the process of designing the precast concrete frame, must be listed, together with details of the specific information they require or are to provide, and the dates by which the information is to be available. The schedule should relate to the production and erection programme and should enable the consequences of late information or delays in making decisions to be seen easily. The more precise and definitive the information release schedule, the more useful and effective it will be.

As noted earlier, precasters should become effective members of the design team from the day of their appointment. The information release schedule establishes the ground rules for the extended design team and forms the basis for monitoring the design. There should be a programme of regular meetings, at which progress is checked and design problems discussed. The need for such face-to-face meetings cannot be too highly stressed. Without them, misunderstanding and ignorance of each party's concerns and priorities will prevail. The meetings should be attended by the precaster and engineer, with the architect, quantity surveyor and services engineer available if required. Later, the main contractor and the precast concrete erector should also be represented.

PRODUCTION

As soon as production begins, most of the engineer's procurement problems should be over, but his duties normally will extend to ensuring proper quality control procedures are being adhered to, and to monitoring the progress of production.

The quality control manual should be studied and all the precaster's operations checked for compliance. Record sheets should be available for each unit cast and should provide a detailed history of it. The majority of precasters have BS 5750 accreditation, which obliges them to implement strict control procedures (see Section 5).

After casting, units will be stored to await despatch to site. It is normal for the contract to require payment for items in stock, so it will be necessary for these to be inspected and counted regularly, for example, monthly. All units for which payment is to be certified should be clearly marked with the job name or contract number, an identification number and any other information required (see example specification in the Appendix). The client may also require all units paid for to be marked, for example, 'Property of ABC Developers'.

The sequence of casting in the factory may not reflect the proposed erection sequence. It is more likely to be dictated by the re-use of moulds; all beams from one mould, for example, being cast in a batch, followed by those from the next mould and so on. The precaster will have a target number of units to be cast before commencing erection. This ensures continuity of erection and provides a buffer stock against delivery and site problems. The engineer should ascertain this target figure and note progress towards it during the early stages of production.

Finally, there will be a temptation to neglect the continuation of factory visits when quality control has been proved and erection begins. Quality control of production at the factory must be maintained until the last piece of precast concrete is on its way to site.

DELIVERY AND ERECTION

Delivery and erection are matters for the precaster (including haulier and erector) and the main contractor. Delivery of large numbers of heavy, bulky precast concrete units is a major logistical exercise, upon which the overall contract programme will be heavily dependent. The engineer should not interfere except to ensure that delivery does not affect, and is not impeded by, the design. Many aspects will need to be considered, such as the availability of access routes to the site at all times during construction, and any existing or proposed height, width or loading restrictions.

Precasters normally prefer to erect directly from the transport, thus avoiding double handling with its increased risk of damage to the components. Little, if any, storage area will be required on the site. The responsibility for the inspection, and acceptance or rejection of units when they arrive on site must be established in advance. The criteria for rejection and permissible remedial work must be agreed by the engineer, the precaster and the main contractor.

Erection should follow the previously agreed sequence and method statement closely, and no alterations to precast units should be permitted on site, unless there are exceptional circumstances, and subject to detailed prior agreement with the engineer.

Before erection starts, the precaster's engineers should visit the site to inspect the foundations, holding down bolts, dowel pockets and other column base connections, for line and level. This should be done sufficiently in advance of erection to provide time for any necessary remedial work. Inaccuracies in the foundations are the most common cause of delay to precast concrete frame erection.

Quality control and the progress of site erection of the precast frame will be monitored by the main contractor and the engineer's site supervisory staff – who ideally, should be familiar with this form of construction.

Having reached this position, the role of the engineer in connection with the precast concrete frame is virtually complete. Many building professionals have now gained first-hand experience of the procurement of precast concrete frame construction as described above, and their projects have benefited significantly from it. That experience needs to become more widespread and it is hoped that this Section will serve to accelerate that process. ■

3

GENERAL DESIGN CONSIDERATIONS

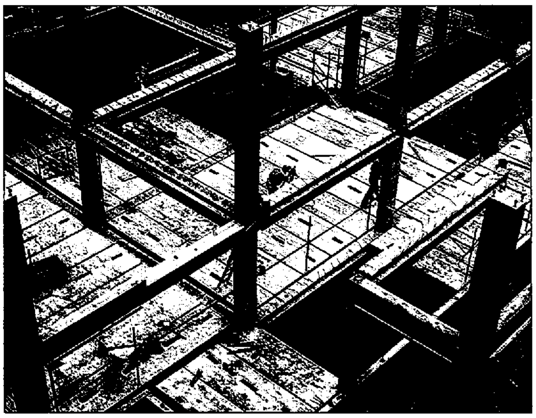
The general considerations in design, as opposed to the detailed design of components, include selection of the frame, the optimum use of components, provisions for services, special features such as cantilevers, and other items requiring specification, such as appearance and finish, tolerances and other performance requirements. For most buildings the selection of internal frame components is governed by the demands of the layout, such as the need for clear floor areas, the location, size and orientation of lift shafts and stairwells, mezzanine floors, and major sub-divisions of the building. The choice of external components is governed by the façade, and the designer is able to specify an external frame that is, in the main, different from the internal arrangement, and to adjust the frame components to suit both internal and external requirements.

SELECTION OF FRAME AND COMPONENTS

The first task is to select the frame components so as to optimize the building design. There are several possibilities open to the designer.

For the internal frame:

- A totally precast concrete frame, consisting of beams and columns, floors, walls, and staircase and lift shaft walls (Figure 3.1).

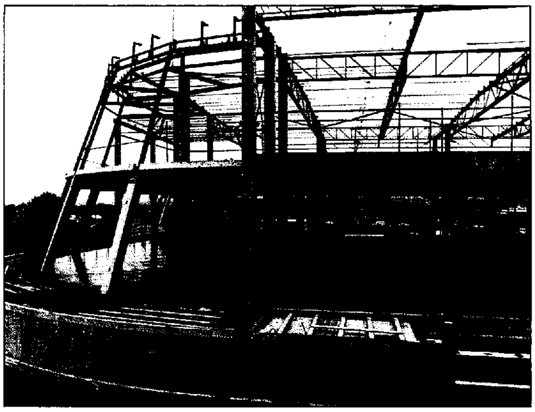


3:1 Typical precast concrete frame building

- A hybrid frame consisting of precast concrete elements designed to act compositely with in-situ concrete or steelwork, or with a combination of these (Figure 3.2).

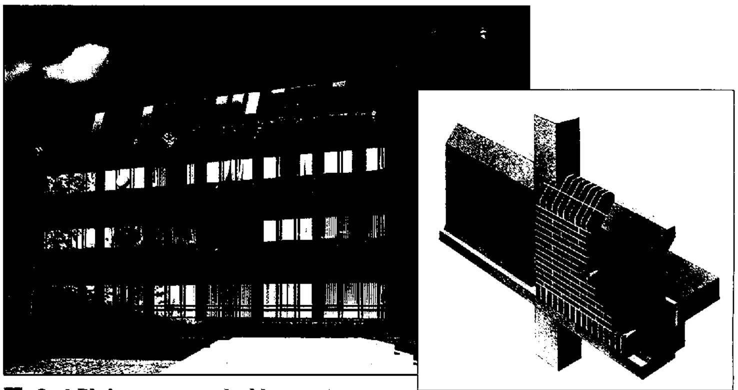
For the external frame:

- Structural elements, such as columns and structural spandrel beams, either of directly finished visual concrete (Figure 3.3), or plain concrete to which cladding will be attached (Figure 3.4).
- Full-height structural cladding units, again either of plain concrete with, in this case, fixings for a subsequent masonry outer leaf (Figure 3.5), or of directly finished visual concrete (Figure 3.6).
- A structural column and beam frame supporting precast concrete cladding, with columns either of plain finish or of visual concrete.
- A structural column and beam frame (Figure 3.7), supporting masonry, metal cladding, glass curtain walling, or other cladding.
- A combination of any of the above, varying around the perimeter.



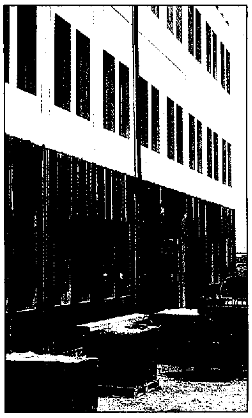
3:2 Precast concrete frame designed to act compositely with in-situ concrete and steelwork

3:3 Loadbearing spandrel beams of high quality visual concrete



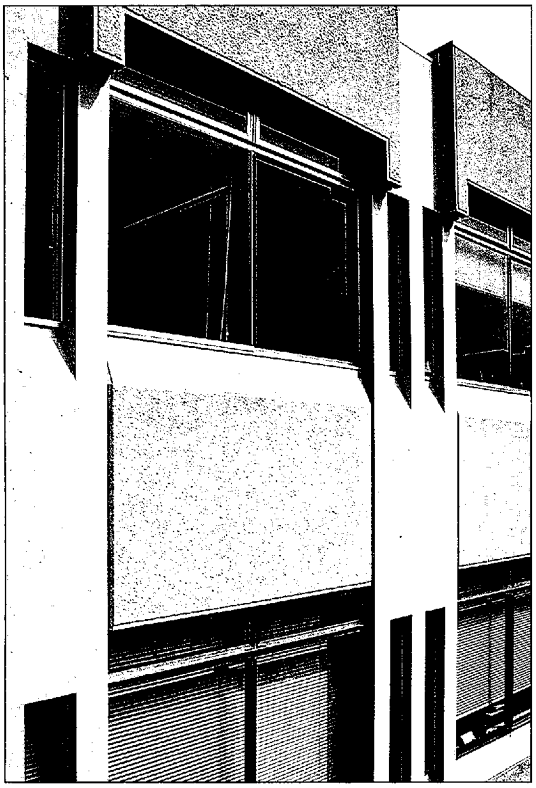
3:4 Plain grey spandrel beams to support external cladding

3



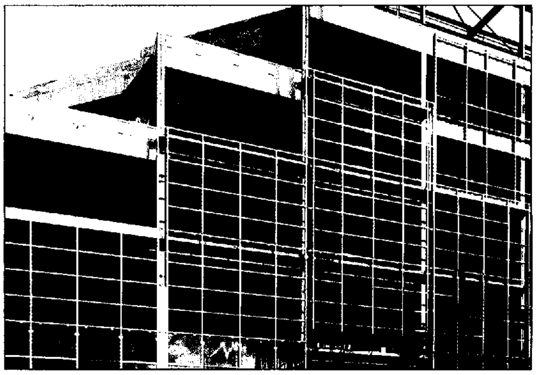
■ 3:5 External loadbearing panels to receive external cladding

In addition to those components where the finish has been specifically mentioned, all components can be made with a high quality finish for those surfaces to be left exposed, or with a plain grey finish as desired.

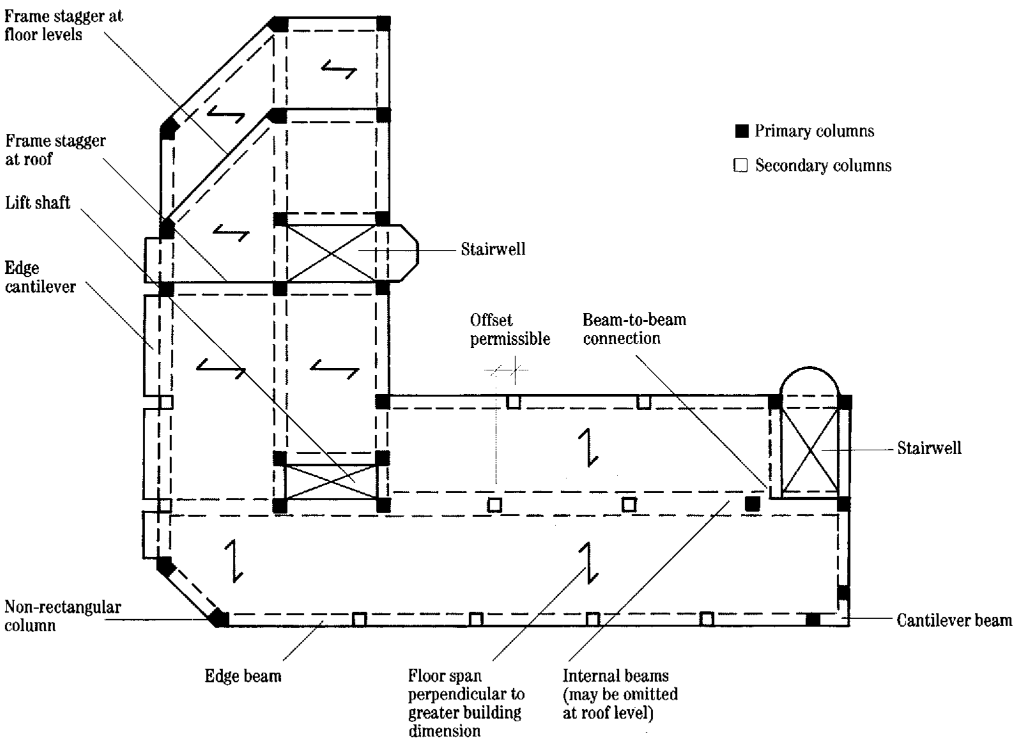


■ 3:6 External loadbearing wall panels produced in high quality visual concrete

The second task is to prepare the framing plan to make the optimum use of precast concrete components. The example in Figure 3.8 shows a building plan sub-divided into bays with repeating shapes and dimensions to give the most economical distribution of beams and slabs. Primary columns are located at strategic positions on the intersecting transverse beam lines, such as at corners in floor level. Some variation in these positions is possible by the use of cantilever beams or beam-to-beam connections. Secondary columns are positioned to satisfy architectural requirements, such as the width of window bays or the position of internal partitioning, or to obtain structural economy by using the minimum number of components to provide acceptable structural floor zones. Their locations are determined by the need for them to support adjacent beams whose lengths may be varied by the



■ 3:7 Precast frame with cladding and glass walling



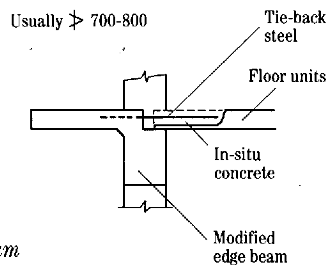
■ 3:8 Example of precast concrete frame layout designer. At certain locations, columns may by replaced by loadbearing wall panels.

Cantilevered balconies

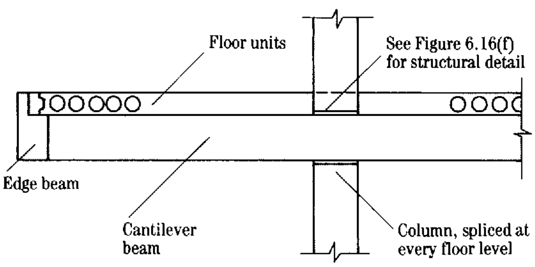
Cantilevered balconies may be formed in a number of ways.

- Edge beams may be modified to provide a 700 mm to 800 mm cantilever (Figure 3.9a). The beam will be a special casting, but the extra cost will be fairly small, particularly where there is considerable repetition. In-situ concrete tie-backs may be necessary, as shown in the figure.
- Beams may be designed to cantilever over columns (Figure 3.9b), to provide a projection up to about 2.5 m, although it can be larger with special or deeper beams. The projection may also be varied to produce changes in plan shape and elevation. Cantilever beams require columns to be spliced at every floor level. This design may also require a greater number of beams and may need an additional edge beam at the end of the cantilevers. However, longer cantilevers can be obtained than by using edge beams with projecting slabs.
- Certain types of floor units, such as double-tees, may be designed to cantilever up to about 1.5 m directly over edge beams. The overall structural zone may then be large because it is not possible to use halving joints in the floor units at the supporting beams, but this is not a problem where the floor cantilevers over a structural wall. Hollow-core units are not recommended for direct cantilever action, other than for small cantilever spans. When using any type of floor, the manufacturer must be consulted because the floors units are normally designed for use only as simply supported spans.
- Tee-columns may be used. These are cantilever beams cast integrally with the columns. Figure 3.9c shows the shape and connection detail of these units. The cantilever is limited by the moment transfer capacity of the columns and is about 2.0 m to 2.5 m for typical floor loadings and external cladding arrangements. The individual nature of these units, and the costs of moulds and transport, means that their cost is likely to be higher than for other designs of cantilever balconies.

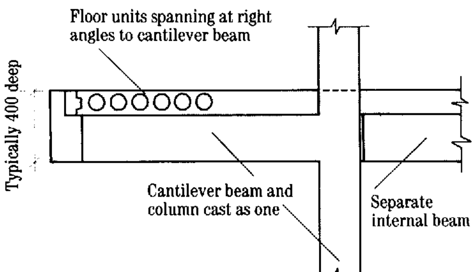
3:9 Cantilevered balconies



a) Modified edge beam



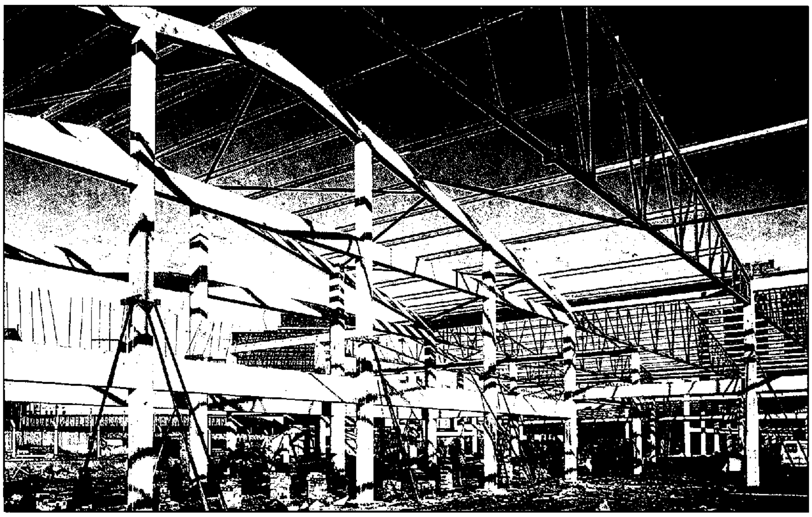
b) Cantilever beam



c) Tee-column

Composite and mixed construction

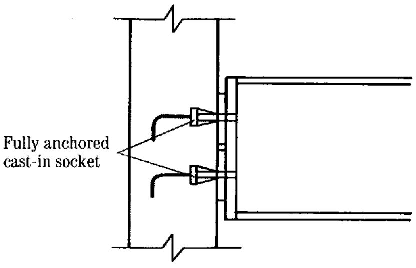
Precast concrete frames are compatible with most other forms of construction, such as in-situ concrete substructures and steel roofs (Figure 3.10). Mixed construction, for example a precast concrete frame with a timber roof, is also sometimes used. Structural masonry is not often combined with a precast concrete frame because the frame is normally erected too quickly for the load-bearing masonry to keep pace. Masonry is, however, sometimes used to provide infill shear walls (see Section 6).



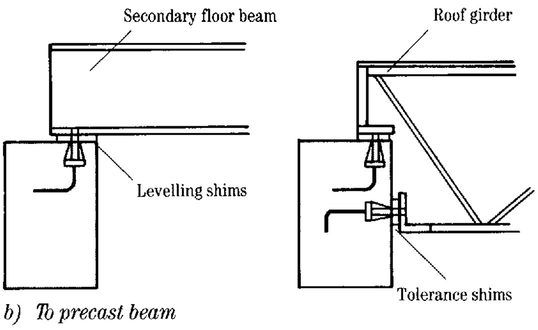
3:10 Precast concrete composite frame

An equivalent steel girder, truss or portal frame may be substituted for a precast concrete component, provided that an appropriate connection detail is used, taking stability into account. Frame components can, therefore, be varied to satisfy both economic and architectural requirements. Fixings for structural steelwork can be accommodated easily in precast concrete components (Figure 3.11). It is often better to post-drill minor fixings.

■ **3:11 Connections for structural steelwork to precast concrete**



a) To precast column or wall



b) To precast beam

Mixed precast and in-situ concrete frames have been used successfully in the UK. Examples are given in references 3,4, and 5. Although this publication is primarily concerned with precast concrete, the details provided may be found useful when combining precast with in-situ concrete.

It is common for in-situ concrete to be used solely for the substructure (e.g. for underground car parks, access ramps and retaining walls). Structural compatibility is seldom a problem. Apart from considerations of stability, it is the design and construction of the joints that require the greatest attention.

Joining precast to in-situ concrete demands particular accuracy in the in-situ work because of the smaller tolerances in the precast units. There is more latitude in joining in-situ to precast concrete, because inaccuracies can be taken up in the in-situ work.

HOLES AND FIXINGS

General

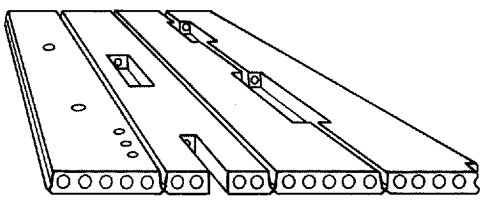
There are no all-embracing rules for providing holes and cast-in fixings in precast concrete components. Their provision and location may need to vary with different methods of manufacture. For example, if the units are cast in steel moulds it may be less expensive and time-consuming to drill and fix on site.

The precast concrete manufacturer should be consulted early in the project to ascertain which holes and fixings can be incorporated during manufacture and which will require attention later. Some general guidelines are given here.

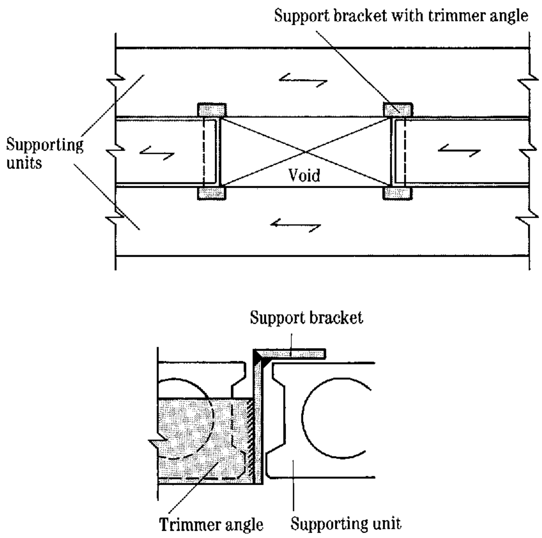
Floor units

Holes, openings and notches in hollow-core and double-tee flooring units are best formed after casting, but before the concrete has hardened. Small holes, up to about 300 mm diameter, are best core-drilled on site. As shown in Figure 3.12a, holes can be provided in a wide variety of sizes and positions, which will be influenced by considerations of structural design and handling and may be readily determined with the precaster. Where the openings are too large to be incorporated within the unit, trimmer angles are used to carry the ends of floor units at the edges of large holes. (Figure 3.12b). The floor units on either side of the hole must be strong enough to carry the additional loading.

■ **3:12 Voids in floors**



a) Voids in hollow-core units

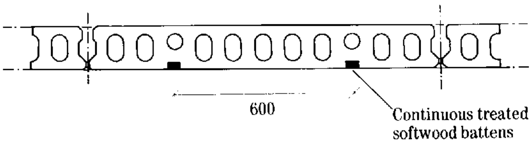


b) Use of trimmer angles for large voids

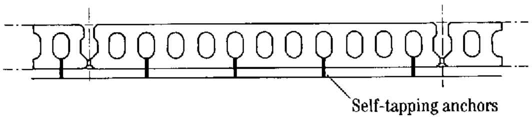
Generally it is not possible to cast sockets or other fixings into the soffits or sides of prestressed concrete floor units, although some manufacturers may be able to provide units with cast-in timber battens (Figure 3.13a). Alternatively, separate battens may be fixed directly to the soffits (Figure 3.13b). Most fixings into these units are formed on site, using proprietary self-tapping anchors, which should be used in accordance with the manufacturer's instructions. Shot-fired fixings are not recommended in prestressed units.

Ceiling hangers (Figure 3.14) are used extensively in commercial buildings where ceilings and electrical and mechanical services are suspended from the floor. The load capacities, spacings and drop of these hangers cover most practical requirements.

3:13 Fixings in slabs

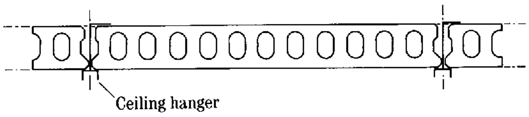


a) Timber battens cast into slab soffit

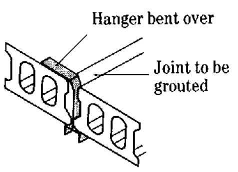


b) Counter battens fixed directly to slab soffit

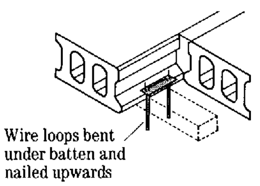
3:14 Ceiling hangers from slabs



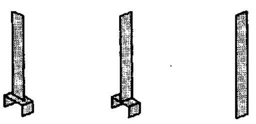
Clips or hangers fixed in slab joints



Ceiling hanger in slab joint



Wire ceiling clip in slab joint



Typical ceiling hangers

Beams, columns, walls and staircases

Methods for providing fixings in these components depend largely on the type of moulds in which the components are cast. In general, units having a preferred cross-sectional profile (as defined in Section 4) are cast in steel moulds, and others are cast in timber moulds. Fixings in concrete cast in steel moulds may be battened in the mould (Figure 3.15). In plain concrete units, the resulting check out in the concrete is not usually a structural or architectural problem. In some cases manufacturers may prefer to drill holes in the moulds and repair these later by plugging and grinding smooth. This may be required for units with a visual concrete finish. In components cast in timber moulds, fixings and holes are limited only by structural considerations. This is particularly true where the fixings or holes are near the main beam-to-column connections.

Many fixings are best drilled on site, but manufacturers can supply a range of built-in fixings. Examples are listed below.

- Cast-in sockets.
- Galvanized steel dovetailed slots for masonry and other types of cladding.
- Dovetailed timber battens for window fixings.
- Chases for weatherproofing.
- Supports for in-situ brickwork, either a bolted on steel angle or a cast-on concrete nib with drip.
- Horizontal service holes (See Section 4 – *Beams*, page 43).
- Internal rainwater pipes.
- Protection against impact and damage (usually at corners).
- Chases for electrical fittings.

In walls, fixings and provisions for services are usually restricted to cast-in sockets, chases and holes. Holes up to 300 mm diameter are easily accommodated, but large holes, exceeding about 500 mm square, may be prohibited from certain structural zones. These restrictions are described in Section 4.

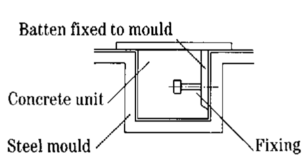
Fixings for balustrades, lighting, and other similar items, may be cast into precast staircase and landing units, but it is often more economical and accurate to drill and fix on site. Stair nosings, and granolithic and non-slip surfaces are more easily added on site, although rebates or rounded edging at the nose can be precast. However, the high quality of concrete in precast units usually makes nosings unnecessary.

APPEARANCE AND FINISHES

Precast concrete frame components can be produced with a wide variety of finishes (see Section 5). These range from reasonably good ex-mould surfaces to high quality visual concrete. Considerable architectural freedom and range of expression can be obtained by using beams and columns with special shapes and with high quality finishes (see examples of buildings in Section 1). Normal or plain finishes will be adequate for most structural components, but high quality surfaces must be specifically specified.

Comprehensive information on types of concrete finishes and methods for producing them can be found in the BCA's *Appearance Matters* series.[6] Appearance and finishes will be important when setting tolerances for frames and components.

3:15 Casting-in a fixing when using steel moulds



FRAME AND COMPONENT TOLERANCES

There will be inevitable differences between the specified dimensions and the actual dimensions of the components and final building. These deviations must be recognized and allowed for. Precast concrete is generally manufactured with relatively small deviations, and very fine specified deviations can be met for special work, but designers should take a realistic view of dimensional variability.

It is essential to consider this from the outset and to discuss tolerances as early as possible with the frame, cladding and services suppliers, to ensure the practicability of the design details in relation to fit. Specified deviations should be appropriate to the design and based on normal construction methods and current practice, so avoiding the imposition of unnecessary constraints on manufacture and erection.

BS 5606[7] gives advice on how to prevent problems of inaccuracy and fit from arising on site. It also gives information on dimensional variations, and their effects on components and on the building. BS 8110[8] gives further examples of dimensional deviations appropriate to precast concrete components.

For a precast concrete frame, the important factors are whether it is to have a plain or a special finish, and the type of cladding which it is to support. Other deviations which usually have to be taken into account are those induced during setting-out and construction, camber, and deflections caused by dead and imposed loads.

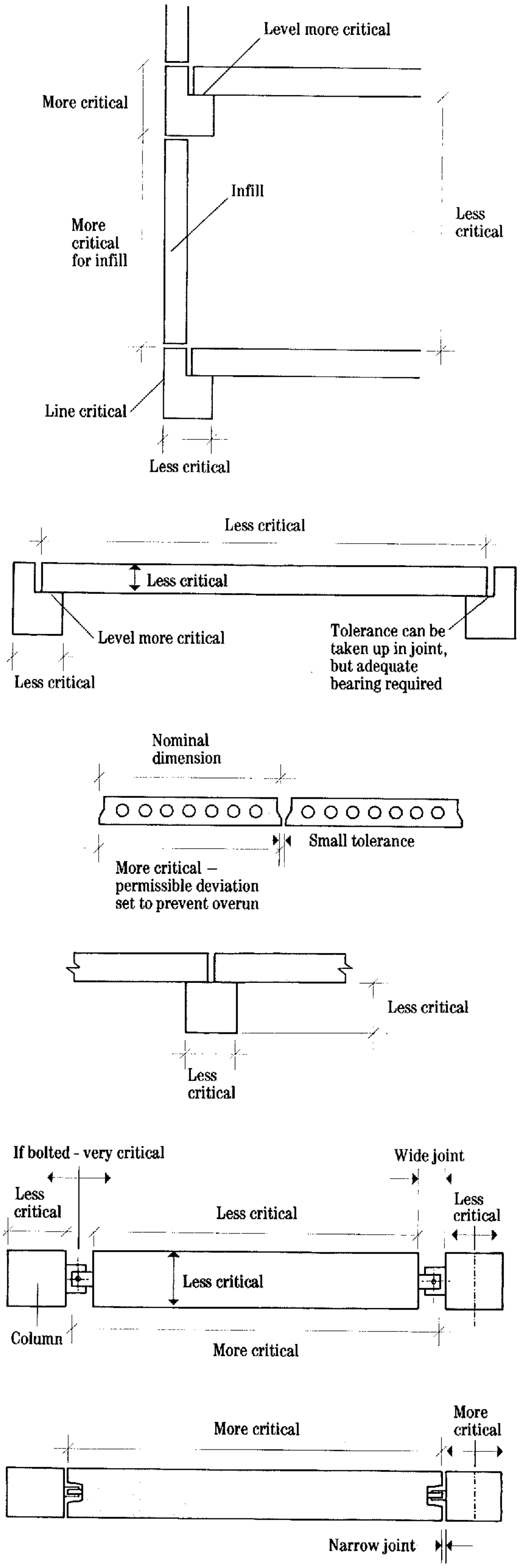
A number of general factors which relate to buildings as a whole, and a procedure for specifying permissible deviations, are given in BS 5606. This emphasises the need to identify locations where dimensions are likely to be critical, to choose design details which avoid problems of fit and minimize the number of tolerance constraints, and to select reasonable and achievable tolerance targets when it is not possible to avoid tolerance problems by design.

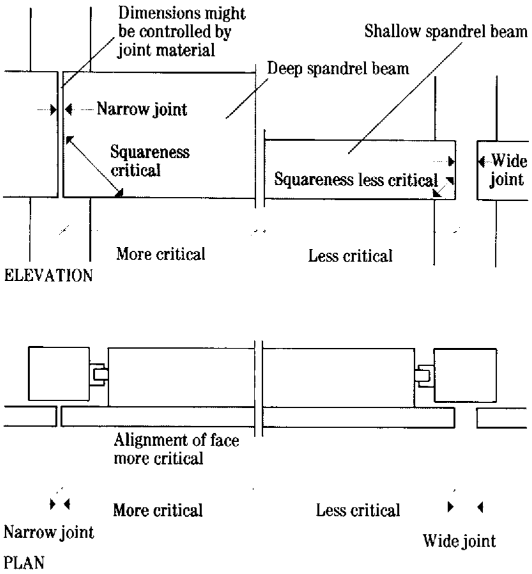
For many frames and the majority of components, it should be sufficient to specify permissible deviations to the level of accuracy given in Tables 1 and 2 of BS 5606.

Specifying very small permissible deviations for all dimensions is unlikely to be reasonable or necessary. For example, the permissible deviation on the length of an extruded floor slab, or on a beam bearing on a steel connection or corbel, would not need to be as small as that required for an exposed spandrel edge beam, or for fixing cladding.

For dimensions where smaller permissible deviations are deemed necessary, they will have to be specially determined and additionally specified. The word 'additionally' is used purposely, because the smaller values only apply to some dimensions. Figure 3.16 shows examples of dimensions of differing criticality, to demonstrate the need to pay attention to particular details rather than adopting a stringent general specification. The specification should be accompanied by drawings showing clearly to which dimensions these smaller values apply.

3:16 Criticality of frame and component dimensions





The manufacture and erection of the frame should be monitored to ensure compliance with the specified permissible deviations. Methods for measuring buildings and building products are given in BS 7307.[9] Table 3 of BS 5606 indicates the accuracy obtainable when using measuring instruments, and the approach to determining the effects of combined deviations.

OTHER PERFORMANCE REQUIREMENTS

In addition to ensuring adequate structural performance, other factors such as thermal performance, sound insulation and, possibly, vibration must be considered.

Thermal performance

Concrete provides a small amount of thermal insulation but, in most instances, additional insulation will be needed to enable the external elements of the construction to meet the requirements of the Building Regulations.[10,11,12] This insulation will normally be provided by the external walls (masonry, precast concrete, metal-backed or glazed screen cladding) or by insulation applied to, or combined with, the walling, as with sandwich panels. Roofs and exposed floors will usually need additional external or internal insulation.

Investigations at the Building Research Establishment have shown that hollow cores in floors may be used as warm air heating ducts. There are limitations in this system, particularly at junctions with beams where the cores are often filled with in-situ concrete.

Sound insulation

Sound insulation between dwellings is a mandatory requirement of the Building Regulations. In other buildings there is seldom any requirement for the control of noise between adjacent rooms, apart from possible requirements of the Factory Inspectorate or similar body. In many cases sound insulation is the responsibility of the designer or client.

The control of sound in buildings is dealt with in detail in BS 8233.[13] This also provides guidance on the level of sound insulation appropriate for different types of building, and on the notional sound insulation properties of a range of wall and floor elements.

To prevent an excessive level of noise within a building, both sound transmission and sound absorption must be considered.

Impact and airborne noise can pass from one location to another by sound transmission along a variety of direct and flanking transmission paths. Such sound can be controlled by isolating it, or by increasing the mass of the wall or other barrier to its passage. An indication of the sound insulation provided by the mass of concrete components is given in BS 8233.

The mass provided by some concrete components is given in Table 4.1. This mass typically provides sound insulation of 45 to 52 dB.

Impact sound transmission can be reduced by a floating screed or resilient surface layer, or by a raised service floor.

Sound absorption is the ability of a building element to reduce the reflection of incidental sound, and control the noise within a room or other enclosure. Concrete generally does not have high sound absorption properties. Where additional sound absorption is required, it can be provided by acoustic ceiling panels and absorbent floor surfaces.

Vibrations

The mass of a concrete structure will also be beneficial in controlling the effects of vibrations from internal plant or from external sources. In extreme cases, buildings may have to be founded on special anti-vibration seatings, but even when this is done, mass is still important.

TYPES OF FRAMES AND COMPONENTS

■ ■ This section gives information on typical structural components for both precast concrete and composite frames (Figures 4.1 and 4.2). The preferred shapes and sizes of precast concrete components are indicated, but as most beams and columns are cast in timber moulds, a variety of shapes can be produced. Some examples are shown in Section 1, pages 9, 10 and 23. General details are given here of the main frame connections, and further information is given in Section 6.

Other components, such as non-structural precast cladding, are outside the scope of this publication and are fully documented elsewhere.[14,15,16]

COLUMNS

It is preferable, but not essential, for columns to be positioned on a rectangular grid so that components will have square ends. These are more economical to manufacture than skew-ended components. Columns generally require a minimum cross-sectional dimension of 300 mm, to accommodate the column-to-beam connections, but a smaller size may be possible. The 300 mm dimension provides a two-hour fire rating, making it suitable for a wide range of buildings. Preferred increments in size are 50 mm or 75 mm, on one or both faces, up to a maximum of, typically, 450 mm by 600 mm.

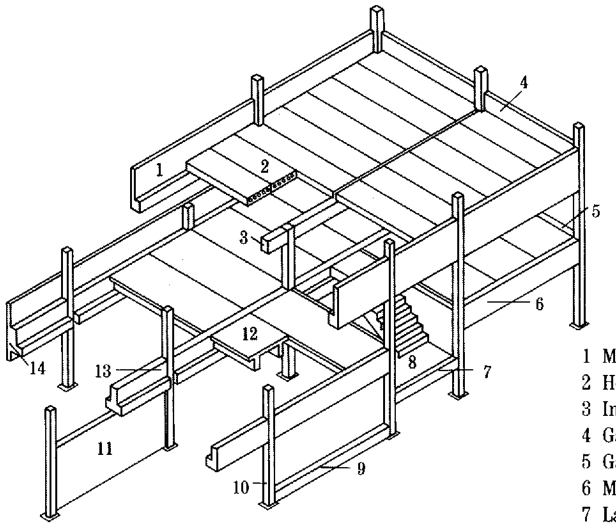
Non-rectangular columns can be manufactured to suit irregular grids and to enable special column profiles to be featured (Figure 4.3).

Columns with a maximum length of 20 m to 24 m can be manufactured and erected in one piece, i.e. without splicing, although the normal practical and economic limit is about 15 m. Storey-height columns with flying-over beams can also be used to good effect, particularly where edge cantilevers are required. Columns may either be plain grey or have a decorative finish on one or more faces. The designer may need to allow for finishes when deciding on the net cross-section to be used in the structural analysis.

Columns may be continuous to the full height of the building, or may be stepped back at an intermediate level to satisfy architectural demands. As in any form of construction, it is desirable to keep columns in vertical alignment and it is preferable to terminate columns at positions where the floor or roof construction can span over the columns omitted beneath.

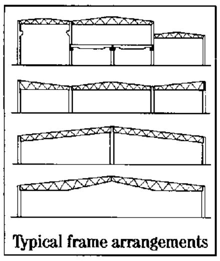
Columns are commonly founded in pockets in in-situ concrete foundations, or on to prepared pad foundations with projecting reinforcing bars or holding down bolts — the base plates can be offset if necessary (Figure 4.4). The column foundation detail may vary according to the manufacturer's

■ **4:1 Precast concrete frame components**

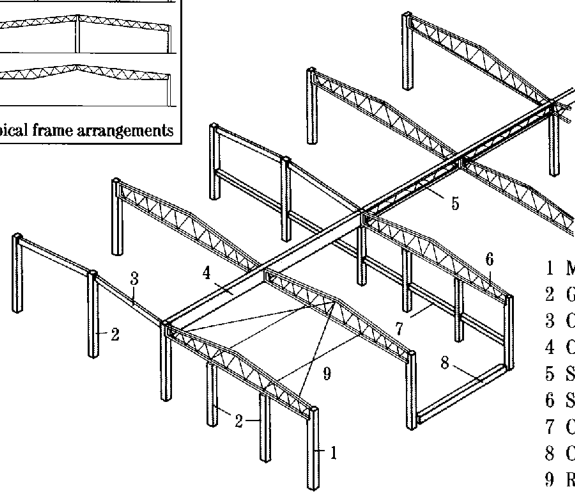


1 Main spandrel beam
2 Hollow-core unit
3 Internal rectangular beam
4 Gable spandrel beam
5 Gable beam
6 Main edge beam
7 Landing support beam
8 Staircase and landing
9 Ground beam
10 Column
11 Wall
12 Double-tee unit
13 Internal beam
14 Main edge spandrel beam

■ **4:2 Composite frame components**

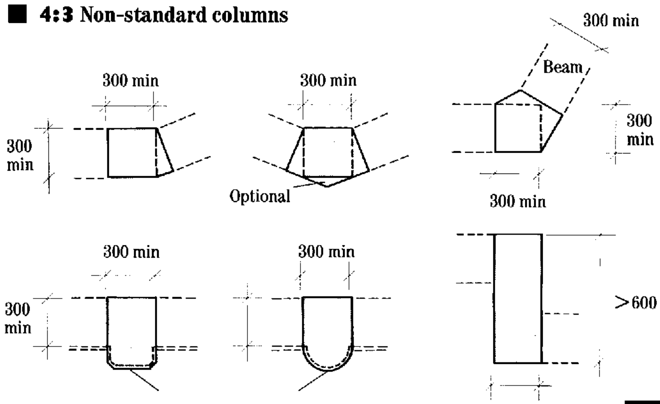


Typical frame arrangements



1 Main column
2 Gable column
3 Concrete raker beam
4 Concrete spine beam
5 Steel spine beam
6 Steel lattice beam
7 Concrete splitter beam
8 Concrete ground beam
9 Roof bracing

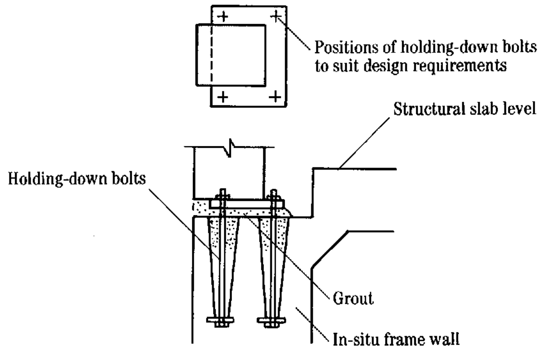
■ **4:3 Non-standard columns**



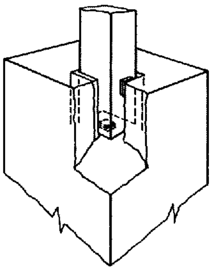
preference or the design requirements. Comments on the more commonly used details are given in Figure 4.5.

At floor levels, columns have structural inserts or corbels to provide bearings for the beams. The positions of the inserts or corbels may be varied to provide connections at different levels on each face of the column, but it is preferable and more economical to keep these variations to a minimum. Reasonable changes in the dimensions or shapes of column cross-sections can be produced, either in a single precast unit or by splicing different sections together.

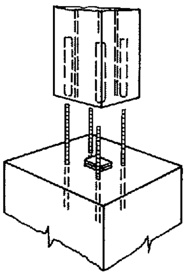
■ 4:4 Offset base plate on in-situ support



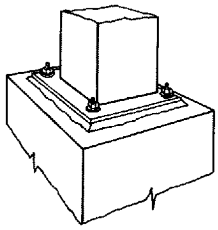
■ 4:5 Column foundation details



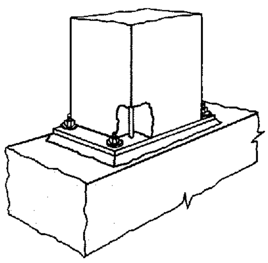
- Moment resistance at column base.
- Minimum tolerance problems.
- Allows continuous column moulds to be used.
- Foundation work deeper.
- Care required to assure good grouting under column.



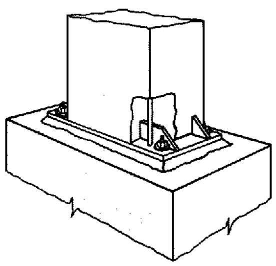
- Moment resistance at column base.
- Connection is concealed.
- Often used for architectural columns where base is exposed.
- Care required to prevent reinforcement from being bent.
- Particular care is required to ensure sheaths and gap under column are properly grouted.
- Care needed in positioning of starter bars and confining links.



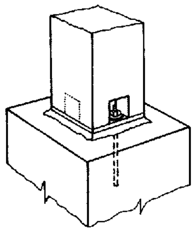
- Column corner reinforcing bars can be welded to plate for anchorage.
- Larger base plate increases effective bearing area.
- Oversize holes reduce tolerance problems.
- Usually requires thicker base plate.
- Large base plate may interfere with existing or future wall located near column line.



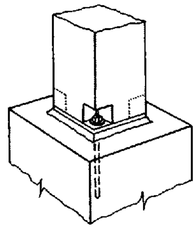
- Main reinforcement can be welded to base plate.
- Oversize holes reduce tolerance problems.
- Large base plate increases effective bearing area.
- Requires thicker base plate.



- Stiffeners allow a thinner base plate, and increased moment capacity.
- Main reinforcement can be welded to stiffeners for anchorage.
- Larger base plate increases effective bearing area.



- Side recesses allow corner reinforcing bars to be welded to base plate.
- Oversize holes in base plate usually reduce tolerance problems.
- Column-size base plate allows continuous column moulds to be used.
- Permits thinnest possible plate.
- Bolts may be used on one axis only to provide semi-pinned joint.
- Side recesses restrict wrench movement.
- Anchor bolts provide less moment resistance.



- Corner recesses allow easier wrench access and effective placement of anchor bolts.
- Oversize holes in base plate reduce tolerance problems.
- Column-size base plate allows continuous column moulds to be used.
- Corner recesses may be formed by steel angle.
- Steel angle may be welded to base plate and column corner reinforcing bar.

BEAMS

A wide range of beams is available in preferred section sizes (50mm or 75mm increments) together with an infinite range in non-preferred sizes. As an example, the illustrations in Section 1 shows buildings with differing beam profiles, including some with curved soffits. Variations in profile and span are limited only by aesthetic and budgetary constraints.

Internal beams may be prestressed or reinforced. The most common shape is the inverted tee (or double boot) (Figure 4.6a), where part of the beam section falls within the slab so that the whole depth of the available floor zone is used structurally. The width of the boot is governed by the need to provide an adequate bearing for the floor slab.

Rectangular beams (Figure 4.6b) are less efficient structurally than inverted tee-beams, but they are simpler to manufacture so may be more economical. They may be considered where the structural zone is not a limiting factor.

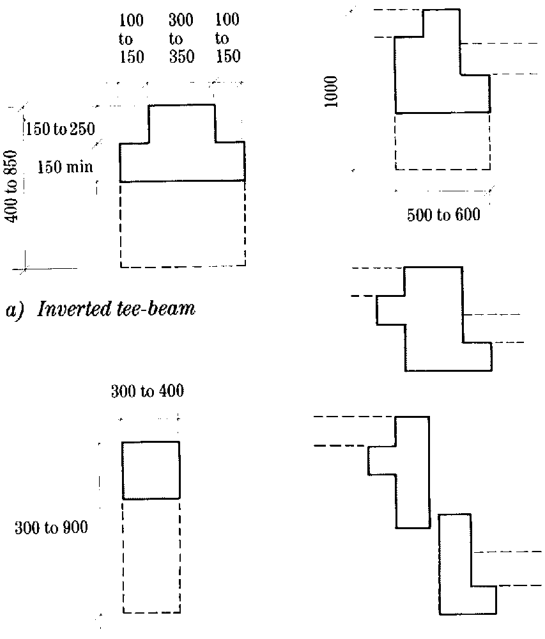
Changes in floor level (Figure 4.6c) may be accommodated by L-beams (or single boot beams), or by building up one side of an inverted tee-beam. Where the difference in floor levels exceeds about 750 mm, a solution is to use two L-beams back to back and separated by a small gap (say 100 mm). This is often used at the split level in car parks, but particular consideration then needs to be given to transverse ties across the structure. The centre line of internal beams is often designed to correspond with the centre of the columns, but the beams can be off-centre where this is required architecturally. The outstand boot on the beam is generally not continuous past the faces of the columns.

External beams are more often made in reinforced concrete because they normally have an asymmetrical cross-section, which makes them difficult to prestress without causing unwanted deflections. Preferred sections vary with different manufacturers, but a typical range is shown in Figure 4.7a. Minimum dimensions are again often dictated by the size of the connection with the column. The upstand contributes to the strength of the beam and acts as permanent formwork for any in-situ infill concrete used with the floor slab. The depth of the upstand may be varied indefinitely.

L-beams with large upstands are known as spandrel beams. They are structurally efficient and help to provide following trades with some weather protection. The width of L- or spandrel beams may be confined within the width of the column, or may project forward of the column to form an outstand spandrel, as shown in Figure 4.7b.

Architecturally, spandrel beams provide the building with a continuous façade, of any practical height, free of columns or other obstructions. Vertical joints between outstand spandrel beams may be pointed or sealed using a cold mastic. Corners may be formed in one of the ways shown in Figure 4.7c.

■ 4:6 Typical dimensions of internal beams

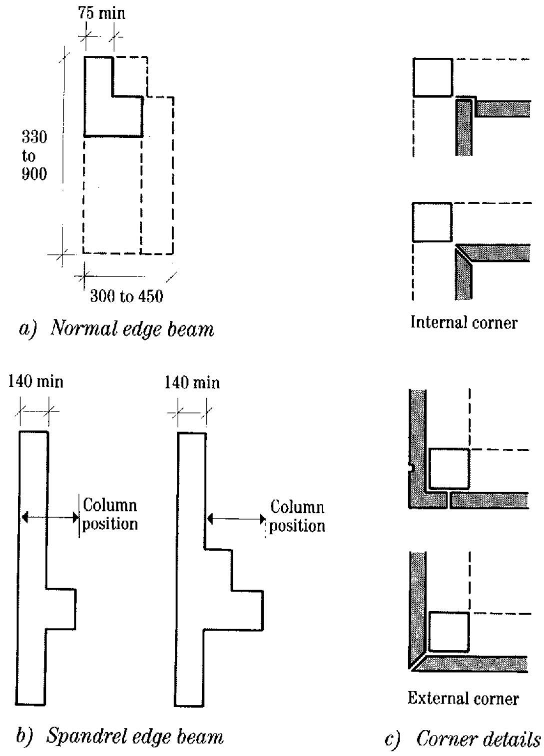


a) Inverted tee-beam

b) Rectangular beam

c) Changes in floor level

■ **4:7 Typical dimensions of edge beams**



The range of decorative finishes is as wide as that for cladding itself.[6,14,15,16] Horizontal service holes may be provided transversely to the span of the beam and at various positions, but preferably they should be located near the neutral axis of the beam and at the one-third points of the span. Individual manufacturers should be consulted about the maximum size and position of the holes early in the design appraisal.

If a precast or other suspended ground floor is specified, it can be supported on precast concrete ground beams which may be architecturally similar to upper L- or spandrel beams. Supporting connections may be made directly to the precast columns, or by dowelling to the foundations.

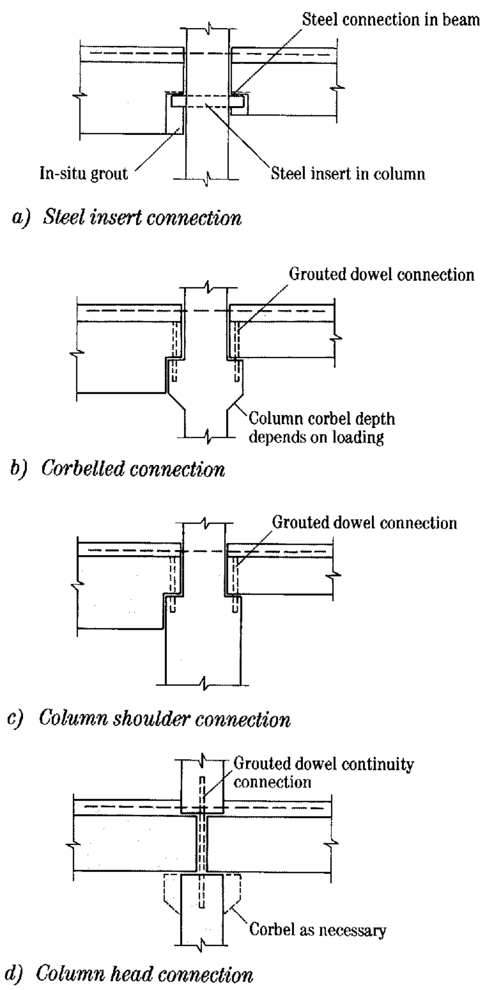
Roof-level parapet beams may be manufactured as single structural units, with horizontal chases for tucking in weatherproofing, or as separate beam and parapet panel units. The panel is then non-structural except for its wind load-carrying capacity, and is attached to the precast columns. The depth of the columns may be equal to the thickness of the panel (typically 150 mm to 200 mm) to form a flush face on which following trades may work.

Some structurally important beams have been made with lightweight aggregates, but their use is limited by the lower concrete strength, particularly at connections. To reduce weight, lightweight aggregates have been used in the upstands of deep spandrel beams, i.e. away from the structural connections, but this practice needs particular care because of the disruption to production and the effects of differential movement between lightweight and normal weight concretes.

One of the most important connections in multi-storey precast construction is that between beams and columns. The connections are discussed in greater detail in Section 6. They are generally assumed to act as pinned joints, i.e. the beam is assumed to be simply supported with no moment transfer between it and the column. However, research[17,18] indicates that many beam-to-column connections have a degree of fixity which could be allowed for. Thus, there is often an untapped reserve of strength, adding to the beam strength and frame stability, and the beam-to-column connections may be modified to provide direct moment capacity where required (see Section 6).

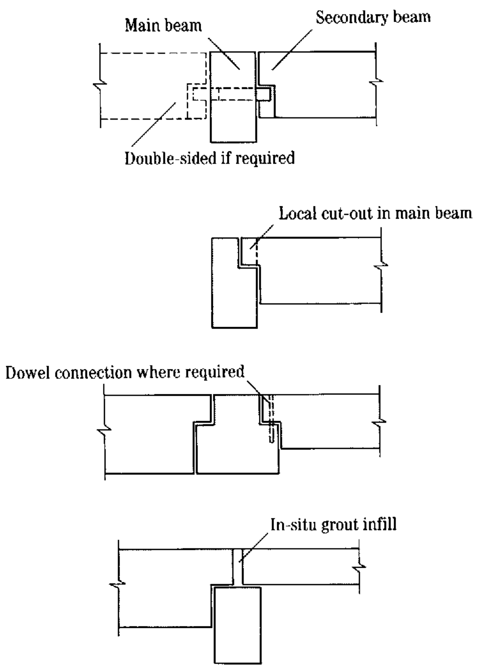
Shear forces are transferred by direct bearing between steel inserts in the beam and column (Figure 4.8a), between the beam and a concrete corbel cast onto the face of the column (Figure 4.8b), or by direct bearing onto a column shoulder (Figure 4.8c) or column head (Figure 4.8d). Up to four beam connections may be made at any one level when using inserts or corbels, but two connections is the practical limit at the haunch or column head.

■ **4:8 Beam-to-column connections**



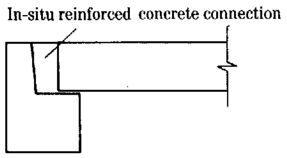
All steel inserts are protected against fire and exposure by in-situ grout (sometimes a non-shrinkable grout) or concrete between the beam end and column face. It is normal to support beam ends on columns but, where it is unavoidable, a direct connection may be made between the end of a secondary beam and a main beam (Figure 4.9). This detail requires special attention, particularly in the design of the main beam, because this type of connection has a restricted shear capacity.

4:9 Beam-to-beam connections

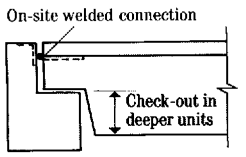


Beam-to-floor connections are also usually assumed to act as simple pinned supports, despite the presence of reinforced in-situ concrete strips, or welded connections between beam and floor, as shown in Figure 4.10. Hollow-core and solid plank floor units are usually laid directly onto the shelf of the beam, but double-tee floor units are laid on neoprene bearing pads. These bearings are discussed in greater detail in Section 6.

4:10 Beam-to-floor connections



a) Beam to hollow-core unit



b) Beam to double-tee unit

WALLS

Precast concrete walls may serve three functions:

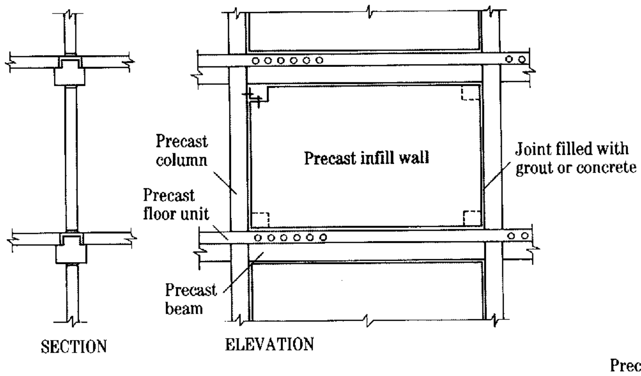
- To provide stability (shear walls).
- To carry vertical loads.
- To act as a fire barrier.

External walls, or walls surrounding staircases and lift shafts are typical examples. The basis for the design of shear walls varies with the geometry of the frame. Their function is to brace the assumed pin-jointed precast frame against side-sway, so relieving the column foundations of overturning moments. Shear walls may be classified as infill or cantilever. Section 6 gives a detailed description but, briefly, infill walls (Figure 4.11a), are designed to act compositely with the beam and column frame, whereas cantilever walls (Figure 4.11b), which may be joined together to form boxes (Figure 4.11c), act as deep beams, which are used to brace the remaining column frame.

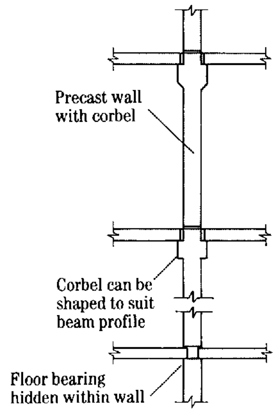
If the opening in the frame is particularly large, say 8 m x 4 m, it may be necessary to 'stitch' two separate wall units together using a vertical reinforced in-situ concrete joint. The thickness of the wall units varies between 150 mm and 300 mm and is governed by the lifting capacity in the factory, or by the shear capacity in service.

The design considers that infill shear walls act as diagonal compressive struts, so generally it is necessary to avoid large holes, such as those for windows and doors, on the diagonals of these units.

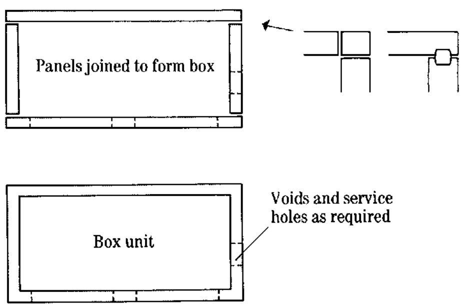
4:11 Shear walls



a) Non-loadbearing infill wall



b) Cantilevered loadbearing wall options



c) Shear boxes in plan

The walls may be pierced by small service holes, up to about 200 mm in diameter, provided that they do not significantly intersect the diagonals or interfere with the structural continuity at the edges. In some situations, however, it may be possible to design shear walls with large openings. Diagonal bracing, which permits significantly larger service holes, may be used as an alternative means of providing stability. This is described in Section 6.

Cantilever walls are designed as deep beams, with tension reinforcement to anchor the unit to the foundation and to provide continuity between successive storey-height units. These walls generally replace the other frame components, so corbels for seating floor units or stair landings, and inserts for connections to beams or spandrels, are needed.

Shear boxes (Figure 4.11c), are a variation of the single cantilever wall. They may be formed on site by 'stitching' two or more wall units together, or manufactured in the factory as a complete storey-height, or part storey-height box with door and window openings as required. Twin lift shafts can be manufactured as two separate E-shaped units and stitched toe-to-toe on site. Lift motor rooms and basement pit boxes may be prefabricated to complete the lift shaft. Lift motor slabs may also be precast, but are more expensive per square metre because of their individual nature.

Precast staircase boxes may also be used for stability and to provide access or architectural features to basements, roofs and plant rooms. Their design is different from that of infill walls, and openings for doors and windows may be specified with considerable freedom, following discussions with the precaster. Most structural walls are of solid reinforced concrete. The manufacturing process varies and the walls may not always have a finish suitable for direct painting on both faces (see Section 5). The surface finish classification must be specified during design.

Shear walls should be positioned to reduce torsional effects in the frame. As they are often located around stair wells or lift shafts, other internal or external shear walls may have to be provided to stabilize the structure properly. The number and position of shear walls does not present a particular problem, but they often require consultation between the architect and engineer.

Stability in the temporary state, before the interfaces between wall and frame are grouted, may be obtained by propping, or by clamping or bolting angles or plates to the frame. Design guidance is given in Section 7.

FLOORS

The use of precast prestressed concrete floor units in all types of multi-storey construction is well documented.[19,20,21] Therefore, in dealing with the design of multi-storey precast concrete frames, some duplication of details and data is unavoidable.

In recent years the wide range of precast concrete floors used in precast frames has reduced to three main types:

- Prestressed hollow-core floors (Figure 4.12).
- Prestressed double-tee floors (Figure 4.13).
- Precast composite solid slabs (Figure 4.14).

There are other types, such as the prestressed beam and pot, or beam and block floor, but these are used mainly in low-rise domestic or lighter commercial buildings. Other forms of solid reinforced concrete slabs, or solid prestressed planks may be suitable in some circumstances, but generally they are not competitive in comparison with hollow-core or double-tee floors. Table 4.1 gives an indication of the sizes and weights of the main types of floor. These will vary, depending on the manufacturing process and period of fire resistance. The example given in the table is for a two-hour fire resistance. Load/span tables are given in Section 6.

Table 4.1 Sizes and weights of common floor types

Type	Unit depth (mm)	Unit self-weight (kN/m^2)	Weight with 50 mm structural topping* (kN/m^2)
Hollow-core	400	4.6 - 4.7	6.0 - 6.1
	320	4.2 - 4.3	5.6 - 5.7
	250/260	3.1 - 3.5	4.5 - 4.9
	200	2.6 - 3.0	4.0 - 4.4
	150	2.4 - 2.5	3.8 - 3.9
	100/110	2.1 - 2.3	3.5 - 3.7
Double-tee	800/825	3.9 - 4.9	5.3 - 6.3
	700/725	3.5 - 4.5	4.9 - 5.9
	600/625	3.1 - 4.0	4.5 - 5.4
	500/525	2.7 - 3.6	4.1 - 5.0
	400/425	2.3 - 3.2	3.7 - 4.6
	300/325	2.1 - 2.8	3.5 - 4.2
Flat plank/ in-situ topping	125	2.9 - 3.0	4.3 - 4.4
	100	2.3 - 2.4	3.7 - 3.8
	75	1.7 - 1.8	3.1 - 3.2

* Weight includes allowance for precamber: for each additional 25 mm of topping add 0.6 kN/m^2.

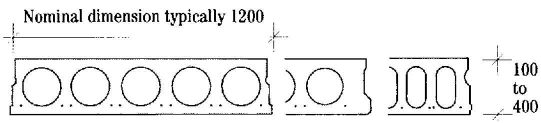
Hollow-core units

Hollow-core flooring is shown in Figure 4.12. The units are manufactured using either a long line extrusion, or a slipforming process, in which the degree of prestress, strand pattern and depth of unit are the main design parameters. Possible variations include increased fire resistance, provisions for vertical service holes, opening of cores for special fixings and cut-outs at columns. Openings and cut-outs are easily formed when the concrete is still 'green', but they are expensive to form later. Such variations or other requirements should be discussed with the precaster early during the design appraisal.

The units are cut to length using a circular saw. A square end is standard, but skew or cranked ends, which are necessary in a non-rectangular framing plan, may be specified.

The edges of the units, shown in detail in Figure 4.12, are profiled to ensure an adequate shear key between adjacent units. The joint should made in in-situ concrete with 10 mm size aggregate, rather than with grout. The extrusion process does not readily permit projecting reinforcement to be provided across the joint. The capacity of the shear key is usually sufficient to prevent differential vertical deflection between adjacent units and to provide horizontal diaphragm action without the use of an in-situ structural screed. This is dealt with in detail in Section 6.

4:12 Alternative cross-sections and edge profiles in hollow-core units



The majority of units manufactured in the UK are nominally 1200 mm wide, although 333, 400 and 600 mm wide units are also available. The actual unit width is usually 3 mm less than the nominal size to allow for constructional tolerances and to minimize over-running of the floor width produced by cumulative errors. To maintain repetition in the floor layout, and hence in the detailing of slabs and beams, the most economical framing plan has column centres on a 1.20 m modular grid.

Units may be readily designed for a one- or two-hour fire resistance and greater fire resistances of up to four hours can be achieved, although these sometimes require protective finishes on the soffit.

Double-tee units

A typical cross-section through a prestressed concrete double-tee unit is shown in Figure 4.13. The main advantages of this type of unit over hollow-core units are:

- Greater span and load carrying capacity.
- The ends of the units can be notched to form a halving joint to reduce the overall structural depth (Figure 4.10b).
- The units are manufactured up to 2400 mm wide (work size 2390 mm), thus reducing the number of units to be fixed on site.

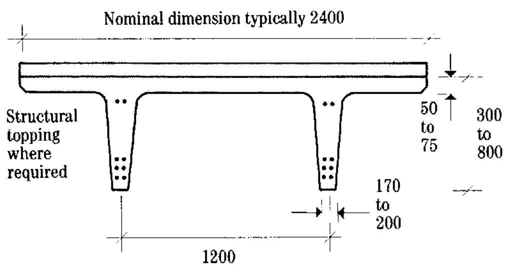
Where units with a shallow flange depth (50 mm) are used, an in-situ reinforced concrete structural topping is required to ensure both vertical shear transfer between adjacent units and horizontal diaphragm action in the floor plate. A positive connection between double-tee units and the supporting beams is achieved by on-site welding. Examples of this are shown in Figure 4.13b.

The standard end profile is square, although contoured ends may be obtained by shaping the flanges to suit the frame. The ends of ribs are generally square. If vertical service voids are required adjacent to supporting beams, forked ends may be formed by setting back the flanges over the full width of the unit as shown in Figure 4.13c. The ribs must be maintained at full length to enable the welded connections to the supporting beam to be made.

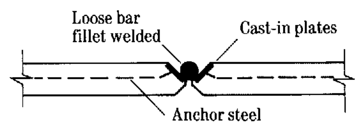
Double-tees may be cantilevered over supports to form balconies, by incorporating top reinforcement so as to modify structural behaviour. It is not possible to use halving joints over cantilever support beams, so a deeper structural zone will be required to accommodate the cantilever. Precast closure pieces for the rib ends can also be supplied. The main problems are in the detailing and in avoiding cold bridging if the units remain physically exposed.

Double-tee units are normally designed to have a minimum fire resistance of one to two hours subject to the correct depth of structural topping or screed being used. Other units with wider ribs and thicker flanges can achieve a four-hour resistance. A greater fire resistance can be achieved on all units by applying appropriate protective soffit finishes. The slabs are manufactured in steel moulds and have a high degree of dimensional stability and an excellent surface finish.

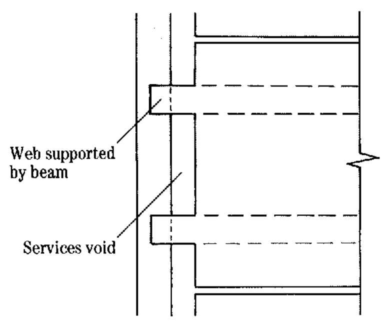
4:13 Double-tee profiles and details



a) Typical double-tee profile



b) Edge connection detail

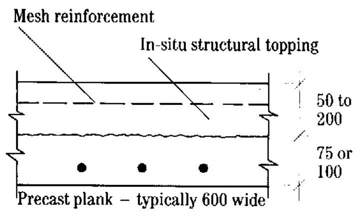


c) Plan detail showing flanges set back for services

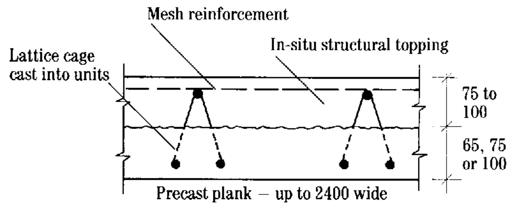
Solid composite planks

Figure 4.14 shows details of precast planks that are combined with an in-situ reinforced concrete topping to construct a robust solid composite floor. This type of floor has the advantage of using units 0.6 m to 2.4 m wide, which have a smooth finish and can be fixed rapidly. However, self-weight is a penalty, as shown in Table 4.1 on page 46, and the planks normally need propping during construction. Service openings up to 2.4 m wide, and cantilevers up to about 1.5 m span, may be formed by reinforcing the in-situ topping. The usual fire resistance is two hours, but thicker precast planks (125 mm) can be used to achieve greater fire resistance, and increased span and load capacity.

4:14 Precast composite solid slabs



a) Prestressed composite floor



b) Reinforced composite floor

STAIRCASES

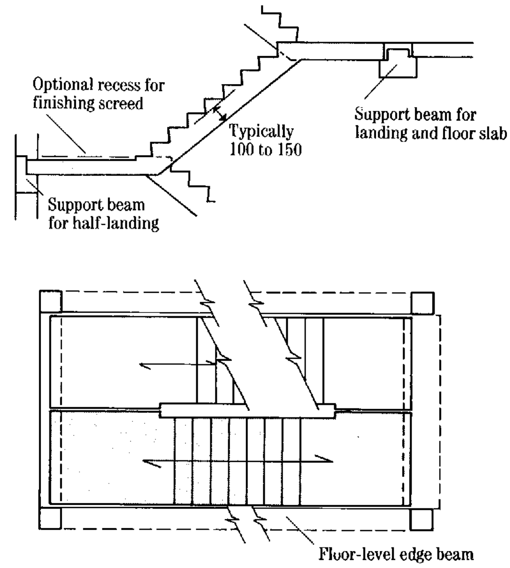
Precast concrete staircases are commonly used with a precast concrete frame and are particularly cost-effective where the design requires a sufficient number of units with a reasonable amount of repetition.

The most common staircase is that described in this section. An alternative is the stringer staircase with separate tread units.

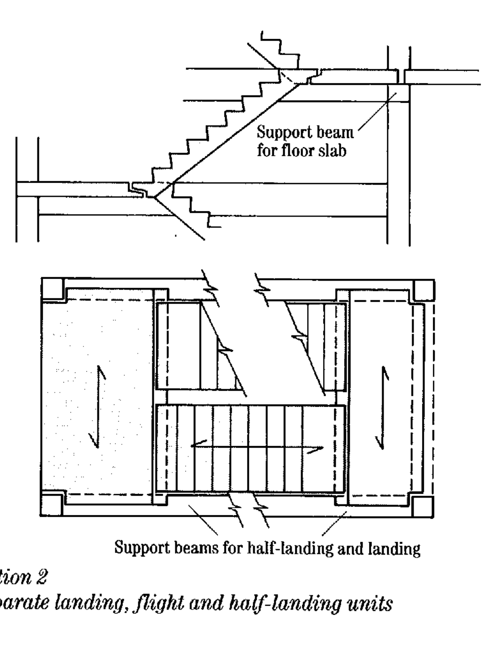
The tread height and going should be consistent within each flight and should also be maintained between flights. The method of manufacture enables the depth of the waist, number of treads and the width of the flight to be varied over a wide range. When using precast concrete stairs, the most important factors are the plan configuration of the staircase and its compatibility with the structural frame. These imply making optimum use of the frame to avoid the need for additional components to support the staircase. Alternative layouts are shown in Figures 4.15 and 4.17 for two-and three-flight staircases respectively.

In Figure 4.15, Option 1 makes optimum use of the frame, because only two short beams are required for support. It is the least satisfactory solution from both structural (it has greater span and depth) and manufacturing points of view. There may be differential levels at floors and half-landings, when either a finishing screed or an in-situ levelling piece is necessary. This would not be the case in Option 2, where optimum use of individual precast flights and landings is made possible by using halving joints to eliminate construction errors. This option, however, requires beams or stub walls, typically 4 m to 5 m long and parallel with the flight, at both floor and half-landing levels. Masonry may be used as a half-landing support, but this enforces a strict programme on both the main contractor and the precast frame erection team. Efforts should be made to make the floor and half-landing supports identical.

4:15 Two-flight staircases



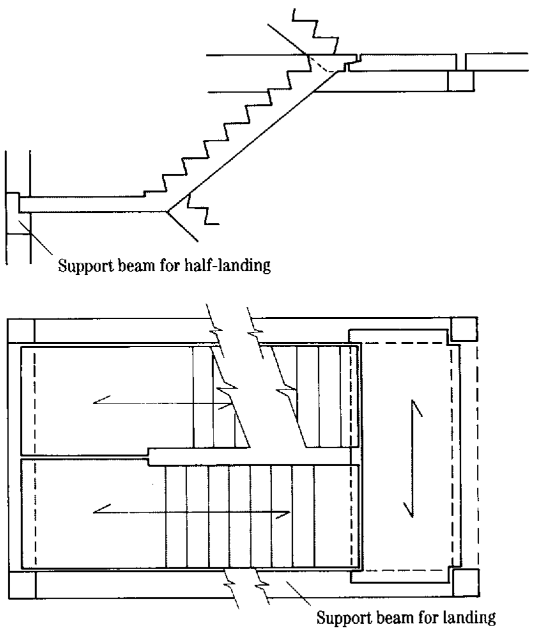
Option 1
Combined landing, flight and half-landing unit



Option 2
Separate landing, flight and half-landing units

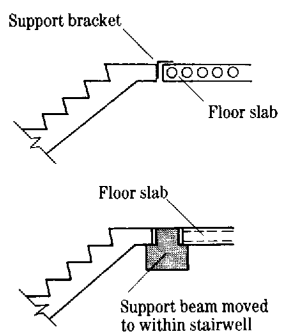
A compromise between Options 1 and 2 is shown in Option 3, where optimum use of frame support beams is combined with an economical use of precast concrete staircase units, which can often be manufactured from one mould. In all cases a minimum depth of unit is required for handling purposes and fire resistance, and this is best utilized in Option 3. A slight variation on this is shown in Figure 4.16, where the floor-level landing is integrated with the floor units and the number of special staircase landing units, which are more expensive than floor units, is reduced. Alternatively the support beam is moved to within the stairwell.

4:15 continued



Option 3
Combined flight and half-landing unit with separate landing unit

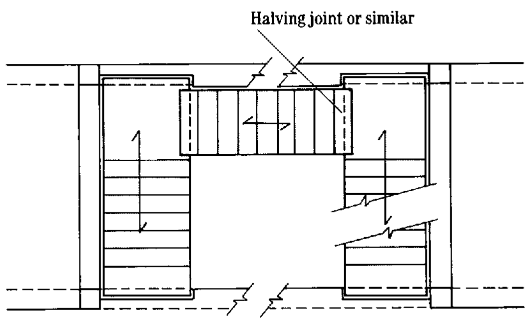
4:16 Variations in flight support



Cantilever half-landings are possible using Options 1 and 3, provided that adequate tie-back is possible. These units may be rectangular, semi-circular or other shape on plan, and may provide support for the cladding.

Figure 4.17 shows a typical arrangement for a three-flight staircase. The dimensions of the well can be varied within the practical limits of the individual flights. The designer has various options, but it is important to minimize the number of precast components whilst maintaining simplicity of manufacture to increase repetition and reduce costs.

4:17 Three-flight staircase



Cantilevers may again be specified, provided that the tie-back is adequate and the support beam is profiled to suit the different landing levels.

The soffits and sides of the units may be specified to have a finish suitable for direct painting; units manufactured on their sides have a good finish on both upper surface and soffit. The units can be designed to have a fire resistance equal to, or exceeding, that for the rest of the frame.

LOADBEARING EXTERNAL WALLS

Loadbearing cladding panels (Figure 3.5), have a dual function in being both decorative and in dispensing with the need for external precast columns, beams and shear walls. Alternatively loadbearing walls to support cladding may be used (Figure 3.4). Both are particularly useful in terms of speed and economy and in assisting to provide a dry enclosure for following trades.

This publication deals with the use of external loadbearing walls integral with a precast frame. Other aspects of the design of precast cladding are described in Reference 14. Where floors are simply supported on continuous seatings corbelled from, or recessed into, the panel, as shown in Figure 4.18, the structural analysis of the system is straightforward. As with all non-monolithic systems, care must be taken when detailing the joints for strength and durability.

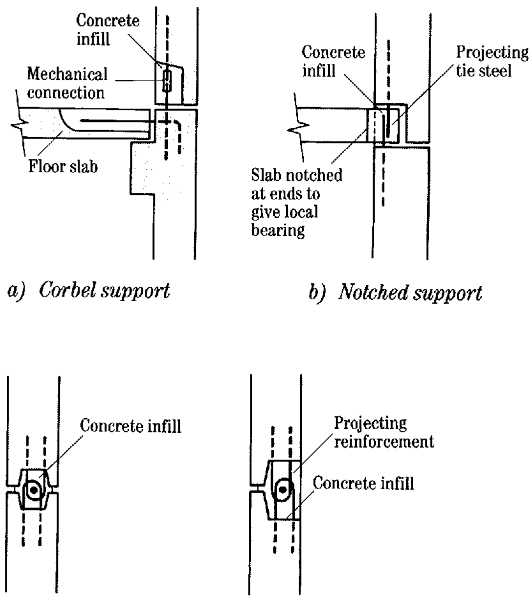
The design of loadbearing external wall panels should aim to produce a compromise between providing the minimum number of panels to fix on site and increasing the self-weight of each panel. The size of loadbearing cladding panels is usually sufficient to present few structural problems, and design may be governed more by the lifting capacity at the works or on site, but severe wind or accidental damage loading may be the decisive factors.

The overall thickness of the panel is determined by its required structural capacity, by the required minimum dimension for the floor slab support, by fire resistance, acoustic or thermal insulation, and the thickness of any decorative finishes. The weight may be reduced by the use of ribbed units, but the effect of stresses caused by the non-uniform cross-section should be taken into account. It may also be necessary to do this with thermally insulated sandwich panels, where ties are required between the separate leaves.

Connections between loadbearing cladding panels and beams are made using steel inserts or corbels. The panels take on the role of columns and must be designed to distribute the large vertical loads at these points. This is particularly important in slender sandwich panels.

Connections between panels and floor slabs are made in the same way as between edge beams and floor slabs. In-situ reinforced concrete strips are cast into hollow-core slabs and welded connections are made to double-tees. The panels can provide lateral stability to pin-jointed frames by effective shear transfer in the horizontal and vertical joints. Figure 4.18 shows examples of how this might be achieved. ■

■ 4:18 Joints in structural loadbearing walls



a) Corbel support

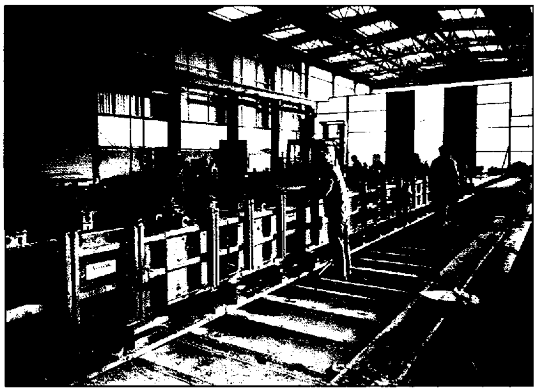
b) Notched support

c) Vertical joints

PRODUCTION

PRECAST FACTORY PRODUCTION

Precasting in factory conditions should ensure the supply of accurate, durable concrete frame components to meet the construction programme. This is because the works environment (Figure 5.1) is unaffected by adverse weather conditions; the components are produced by a stable workforce, which is controlled by experienced managers, supervisors and technicians; and workers are skilled in the assembly of moulds, reinforcement and fittings, and in batching, mixing and placing concrete. This team has the support of experienced mould makers, steelfixers, crane drivers, and stacking and loading personnel.



5:1 Clean, controlled works environment

As well as up-to-date plant for producing batched and mixed concrete to the specified quality, the factory often has specialist workshops for the manufacture and maintenance of moulds, and for the production of jig-built reinforcing cages and connections (Figure 5.2). There will also be workshops for the maintenance of plant and equipment. Storage areas for incoming materials, and stackyards for finished components are matched to the throughput of the works.

5:2 Production of reinforcement cages and main connections

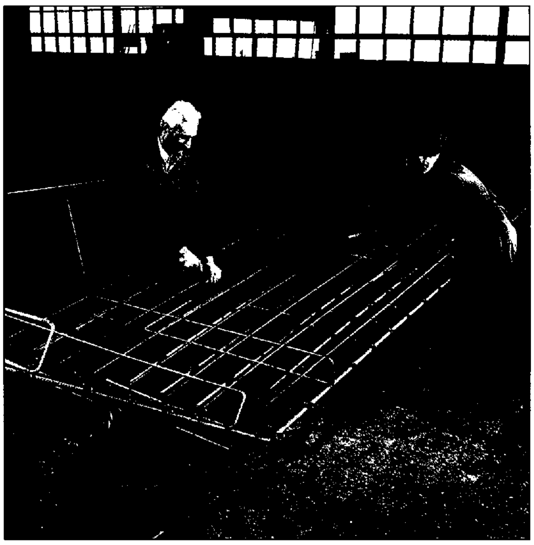
Production programming

The works will organize the production of components in a sequence to meet the requirements of the erection programme, whilst optimizing mould requirements and costs of production. The principal activities of production vary little with the shape and size of the units, so that on discussion with suppliers, architects and engineers designing specialized structures will find that frames containing both special and standard components can be quickly integrated into the production.

Components are produced on a daily cycle, or more quickly where accelerated curing or mechanical production methods are used. When the dates for phased deliveries to site are agreed, the components will be produced in a predetermined sequence to ensure that matured concrete components are ready for instant erection.

Supply of steel reinforcement

Reinforcing cages are fabricated from steel cut and bent in the works (Figure 5.3), or delivered cut and bent from suppliers. Reinforcing cages are assembled in jigs and are subjected to production checks and inspections. The cages are checked for correct cover and location after they are placed in the mould. Concrete cover to the reinforcement is maintained by spacer blocks located at critical positions between the reinforcement and the mould face.

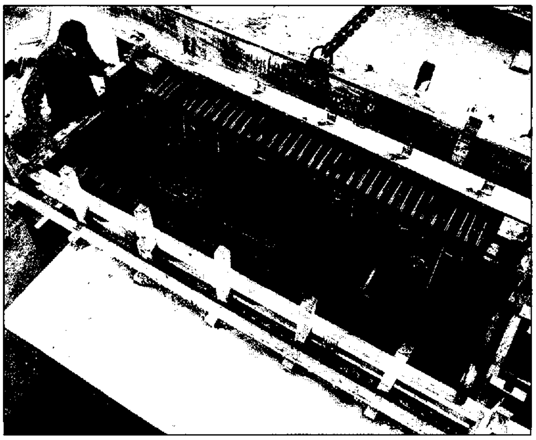


5:3 Assembly of reinforcement

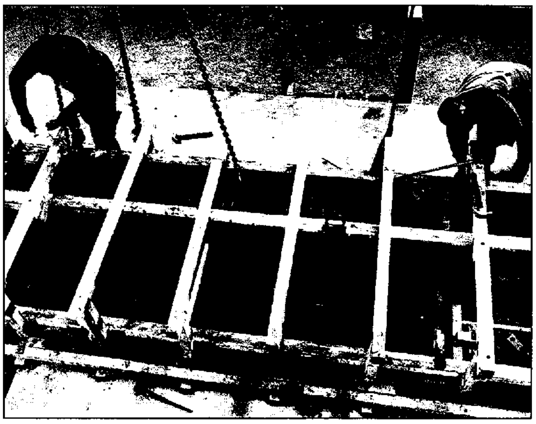
Casting the components

Components can be cast on edge, inverted, or otherwise positioned within the moulds. This gives the manufacturer the ability to choose the method which will ensure proper compaction of the concrete, accuracy and quality of surface finish.

Columns, beams and slabs are generally cast horizontally in moulds mounted on level casting floors (Figures 5.4 and 5.5). Wall panels are cast using either casting tables or tilting frames. Where components are required to have an ex-mould finish on two faces they are likely to be produced vertically in steel 'battery' moulds. Stair flights, and stair and landing units are cast flat or on edge, sometimes arranged back-to-back within the moulds, which thus produce additional ex-mould faces. Lift enclosures and stair walls may be cast vertically around a central fixed mould.



5:4 Reinforcement positioned in partly assembled mould



5:5 Assembly of remaining mould section

Concrete supply, placing and compaction

The capacity of the plant and equipment for handling, placing and compacting the concrete is geared to the quantities of concrete required in a production shift (Figure 5.6).

Many works now employ microprocessor-controlled batching and mixing plants, which store the information for designed mixes of differing proportions for specific applications. These plants adjust the mix proportions to allow for changes in the characteristics of the materials, and record the quantities batched, the mixing time, workability of the mix and other information. Materials are batched by weight with careful adjustment for water content. Admixtures, if used, are measured using calibrated equipment.

Forced-action mixers are normally used to ensure thorough mixing. In some works concrete is placed, spread, and in the case of extrusion and slipform production, compacted by machine. Mixes are called up by a supervisor at or near the point of placing in the mould.

To ensure that optimum density is obtained and that specified concrete strengths are achieved, concrete is placed and compacted using high-frequency external vibrators or pokers (Figure 5.7). The workability is controlled to ensure full compaction.

Admixtures may be used to modify the flow characteristics of the mix, but the precaster should declare these to the purchaser before production starts.

Quality control

Considerable emphasis is placed on quality control in the manufacture of precast concrete. This includes all stages of production, from the receipt of raw materials to the loading and despatch of the finished components. It also includes sampling and testing of the materials and components, and the inspection of the components and equipment, which is carried out by production and quality control personnel.

Sampling and test procedures would typically include:

- Verification of supply, including visits to suppliers works, and routine monitoring of materials combined with scrutiny of certificates from material suppliers.
- Aggregate testing (flakiness index, shrinkage, relative density, water absorption, silt testing and sieve analysis).
- Testing for consistency of mix proportions, and the preparation of test cubes to measure compressive strength.
- Testing for workability.
- Assessment of stripping, handling and transfer strengths by cube testing or using other methods such as a rebound hammer or ultrasonic testing.
- Monitoring the curing cycle and strength development.
- Routine product testing according to the specification.
- Maintenance of test equipment.

Checks and inspections during precasting are made to verify that the moulds are of the required standard, that the reinforcement complies with the schedule and is properly located, and that the cover is correct. Connections and fixings are checked for correct position.

Production checks and inspections include the monitoring of a wide range of activities, including special techniques such as stressing of tendons and the more routine activities of concrete production. Checks ensure that the concrete is of the correct mix proportions and consistency and is properly compacted, and that reinforcement and fittings are not displaced during casting.

Checks and inspections are carried out after casting to confirm the accuracy of the components and the acceptability of the finish. These checks continue until the final inspection of the components loaded on the delivery vehicle.

Many precasters are now registered with nationally accredited certification bodies as 'Firms of assessed quality capability'. Quality assurance systems for the activities of precast concrete design and manufacture were discussed in Section 2.

PRODUCTION TECHNIQUES

Moulds

Moulds are generally made of steel, or timber and ply. They are manufactured for repeated use, to achieve high standards of accuracy and to be capable of generating good quality concrete finishes. Moulds for special structural components are constructed to produce the specified quality.

Moulds are designed and constructed to be capable of casting a group or family of components, and to permit stripping without damage to either the mould or the concrete. Removable pads, fillets and other variable features can be incorporated. If the precaster is given details of these early in the programme, provision can be made for them, thus ensuring that later adjustments do not interfere with the casting sequence.

Connections

Connections, steel billets, and provisions for bolted, swaged or welded connections, are 'transfer-cast' into components during manufacture. Fixings for services and cladding units are located by jigs to ensure the required accuracy.

When transfer-casting, the fixings are held against the mould face so that they keep in place during casting, but when the moulds are stripped they pull away from the mould face and remain in the concrete.

Columns

In the majority of cases, columns are cast horizontally in individual moulds, or in long-line moulds. The moulds are secured to the casting deck to preserve accuracy of line and squareness of section. Connections are located accurately and the concrete is compacted by internal or external vibrators. The method of casting is such that the location of the reinforcement and compaction of the concrete can be monitored continuously.

Beams

As with columns, beams may be cast in individual or long-line moulds. Moulds for reinforced or prestressed components are carefully aligned and secured to preserve squareness and accuracy. Main beam-to-column connections are accurately located. Masonry slots, brick ties, provisions for services, and fixings for plant and equipment are transfer-cast from the moulds into the concrete.

Floor units

Floor units are produced in a variety of ways. Reinforced concrete floor units may be cast in individual moulds or on long-line casting beds, but prestressed concrete floor units are generally cast on long-line beds. The concrete is placed and compacted either manually, or mechanically by extrusion or slipform machines. Standard floor moulds have provisions for producing bearing nibs and connections according to the manufacturer's system. The floor units may be solid or, by using void formers, have a hollow section.

Units may be fully finished on the soffit and top surface, or prepared for a sand-cement screed or structural concrete topping. Prestressed extruded or slipformed floors are manufactured in standard widths and cross-sections, using concrete with virtually zero slump. Special provisions can be made for casting balconies, cantilever slabs and landing slabs for use in conjunction with precast stair units. Wide slab units generally contain longitudinal voids which are formed mechanically during casting.

Prestressed concrete units may be produced by extrusion, slipforms, or manual or machine-casting techniques. Double-tee beams and similar elements for flooring and roofing are wet cast in steel or concrete moulds, either individually or in long-line beds, producing a good, smooth surface finish.

5:6 Placing carefully specified concrete into mould

Stair units

Reinforced or prestressed concrete stair units are cast in standard, bespoke or the variable-geometry moulds which have been developed recently. Straight flights of stairs, or combined stair and landing slabs can be produced. Special finishes, such as granite, non-slip treads, patent nosings and tiles, can be transfer-cast from the moulds, as can fixings for balusters. The method of casting is determined by the type of finish required.

Wall panels

Many loadbearing wall panels are cast in traditional moulds or on tilting tables. Facing materials and special aggregate mixes are generally laid into the bottom of the moulds and the panels cast face-down. This permits special mixes to be placed and compacted, or brick, stone and similar facings to be inserted before placing the loadbearing inner skin. Exposed, plain concrete faces are floated and trowelled to produce the specified finish. Where required, provision is made for installing insulation panels and dry lining. When curing is complete, the panels are turned, handled and stacked vertically to await delivery and erection.

5:7 Compaction of concrete using poker vibrator

Battery casting may be used where large numbers of wall panels are to be cast to a tight programme, or where an ex-mould finish is required on both faces. Battery moulds consist of a series of vertical steel face formers and stop-ends, supported within a gantry, and positioned to generate the required width and edge profile of each wall panel. Battery-cast panels are stacked and transported vertically, or near-vertically. This permits optimization of the reinforcement, which generally takes the form of peripheral bars. Trimming reinforcement is inserted around door and other openings, and loops and anchor plates are inserted for connections. In the case of loadbearing inner-skin panels, fixings are incorporated to support the outer cladding. Provisions for services, switchboxes and similar items can be accurately located by transfer-casting. Pipe and conduit can be cast in.

FINISHES

Frame components are manufactured to the quality and type of finish required by the contract specification. BS 8110[8] describes various qualities and types of surface finish that may be quoted or qualified by the specification (see Appendix, Clause 19). Special surface finishes for very high-quality work are given particular attention.

There are three general types of concrete surface finish.

Finishes from the mould

Concrete frame components will be produced with moulded faces cast in sound, well constructed grout-tight moulds. Generally only minor blemishes are apparent, but any blow holes or similar defects may be filled with cement mortar before finishing by rubbing down with hessian, carborundum or some other medium which tightens the face texture. The amount of subsequent attention given to the surface will depend on its location and the specified quality.

Trowelled and floated finishes

The freshly placed and compacted concrete is levelled by screeding from the mould sides, using manually- or mechanically-operated screeds. Depending on the area of the surface, the concrete is then finished manually, using sleeve floats to achieve a flat, yet open, texture, or by power floating. Where insulation, a screed or a structural concrete topping is to be applied later, the unit's surface may be roughened by tamping, washing and brushing, or by the use of a surface retarder.

Where the component is to have a visual concrete finish, it is usual to finish the surface using steel floats, operated either mechanically or manually. A steel float can produce a flat, tight surface, almost indistinguishable from a moulded face.

Special finishes

Special textured and exposed aggregate finishes are produced by using surface retarders, by washing and brushing, and similar methods. Profiled, featured and similar finishes are produced using mould liners and facings, sometimes combined with subsequent tooling, or abrasive blasting. Details of all these are given in Reference 6. Finishes are produced to match approved samples.

CURING

Components are cured by covering them with insulating sheets to retain moisture and to conserve the heat generated by hydration of the cement. Accelerated curing is carried out by using casting beds and moulds heated electrically, or by steam or hot water. Electric current passed between the steel reinforcement and the mould is also used for accelerated curing. In prestressed concrete production, units are sometimes heated by passing current through the stressing wires.

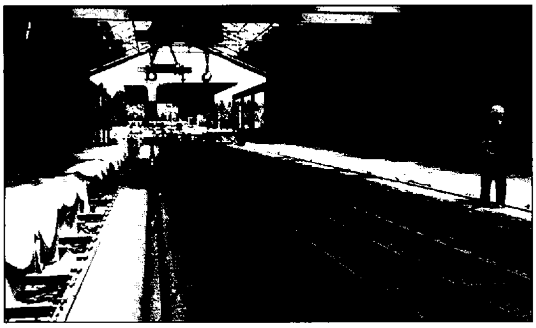
The time at which the concrete has reached the strengths necessary for safe stripping and handling is determined by control cubes cured on the casting bed, or by assessing the maturity, using data from time-and-temperature measuring instruments. Most reinforced components reach a 20 N/mm^2 strength in 24 hours, whilst prestressed concrete units, particularly hollow-core slabs, attain transfer strengths of about 35 N/mm^2 in 12 to 18 hours.

HANDLING AND STORAGE

The simplest forms of lifting are those in which bars are passed through holes cast in the component, or slings are passed around it. The bars must be a good fit in the holes, and washers and pins must be used to retain the bonds or yokes. Slings must be arranged so that they cannot loosen during lifting.

Components may also be handled by lifting connectors cast into the concrete. These connectors may be proprietary studs, loops or similar attachments, or lifting hooks formed from mild steel reinforcing bars. High-yield steel is avoided, although loops of prestressing strand are used for handling large units.

Care must be taken to ensure that stripping and lifting do not damage the fresh concrete. Spreader bars and lifting rigs are used to prevent handling stresses that might damage the components. With the correct handling equipment, which depends on the design of the unit and the type of lifting connection, most reinforced precast concrete components can be handled without damage once the concrete has achieved a compressive strength of about 10 N/mm^2 or the strength necessary for transfer of prestressing. Units are usually handled within hours of casting as part of the rapid production cycle (Figure 5.8).

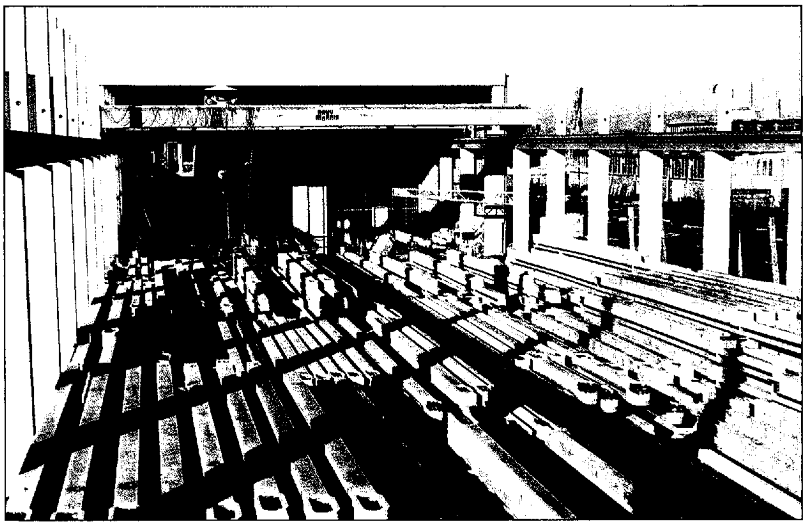


5:8 Prestressed double-tee unit being moved to the storage area

The finished precast components (Figure 5.9) are stacked on clean battens or plastic pads positioned to suit the design of the component. Care is taken to keep the stacks vertical and to ensure that battens are placed directly above one another within the stack.

QUALITY ASSURANCE

Precast concrete manufacturers operate quality control procedures,[22] and many firms are now registered with nationally-established certification and assessment bodies according to a system for the registration of firms of assessed capability. These firms have published their quality policies and have installed quality assurance systems in accordance with the requirements of BS 5750.[23] This requires the personnel responsible for design, production and quality assurance to be named. Management of the quality systems is the responsibility of persons independent of the production line management.



5:9 Storage of high-quality units in works area

Quality systems include scrutiny of the client's requirements for accuracy, surface finish, strength and durability, and the capability of the manufacturer to meet these requirements in producing elements of assured fitness for purpose. Systems are established for the control of design, works instructions and documentation, as well as for quality control measures, testing materials, proof testing in accordance with quality assurance schedules, instrumentation and calibration of laboratory and other test equipment.

Within the registered firm, production is monitored by a series of production checks. Particular emphasis is placed upon training and the establishment of the responsibilities of personnel.

The quality policy, and organization necessary to achieve published standards, of all registered firms operating quality assurance systems, is assessed and monitored by a third party. The assessment also includes all activities from initial contract review and establishment of clients' requirements, through design, purchase of materials and services, to production and quality control, supervision, inspection, testing and training.

Systems are audited and reviewed at specified intervals and the results fed back to the manufacturer to ensure dynamic control and the maintenance of quality standards.

STRUCTURAL DESIGN

BASIS FOR THE DESIGN

Precast concrete frames are often, but mistakenly, considered to consist of individual components which just need to be designed and connected in a manner that ensures adequate strength and interaction between them. However, when designing precast concrete multi-storey structures, the frame has to be considered as an entity. The interaction and relationship between the frame, floors and cladding can be critical to the overall building design, and particularly to achieving a buildable, cost-effective structure.

An assessment should be made of the requirements for each component and its contribution to the building as a whole. This should be done from the start of structural design. When the general and architectural requirements have been satisfied (Section 3), the design may proceed in stages by considering the following aspects.

- Precast concrete components.
- Joints.
- Stability.
- Structural integrity.

Multi-storey precast concrete frames are generally analysed on the assumption that they are pin-jointed braced structures (Figure 6.1a). Stability is provided, usually around staircases and/or lift shafts, by precast concrete or masonry shear walls, cores or shear boxes, or by steel cross-bracing (Figure 6.2). Stability has occasionally been provided by in-situ concrete shear cores or shear walls, but these have not been used extensively because they do not utilize the benefits of prefabrication. They also need additional on-site programming and give rise to shared design responsibilities.

Other methods can sometimes be used to provide stability. For example, in frames up to three storeys, or in three-storey frames with a steel mansard-type roof, it may be possible to consider the frame to be unbraced and to rely for stability on cantilever action in the columns and moment-fixity at the foundations (Figure 6.1b). As an alternative to every column and foundation being made moment-resisting, a small number of large columns, acting as deep beams, may be used as wind posts at intervals along the structure.

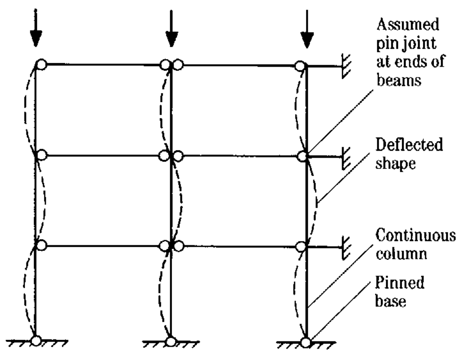
Partially braced frames (Figure 6.1c) offer a compromise between braced and unbraced frames. In some instances stability can be obtained or improved by providing a degree of moment-fixity in the flexural connections between beams and columns.

Although joints are generally taken as being pin-jointed, the tie steel between columns and beams may provide some moment-fixity (Figure 6.2). This will depend on the joint detail, but tension in the ties and compression between the bottom of the beam and the face of the column may develop to provide a stability moment (Figure 6.3a).

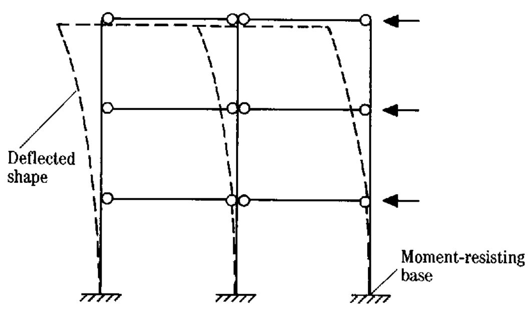
A further modification is to add ties towards the bottom of the beam or by welding or bolting the beam-to-column connection (Figure 6.3b), or by forming a special reinforced in-situ concrete cruciform in the joint between members. In this way moment capacity may be obtained at both ends of the beam.

Other methods of providing stability are discussed later, but it is generally accepted that the most economic solution, in terms of production and erection, is a pin-jointed braced frame.

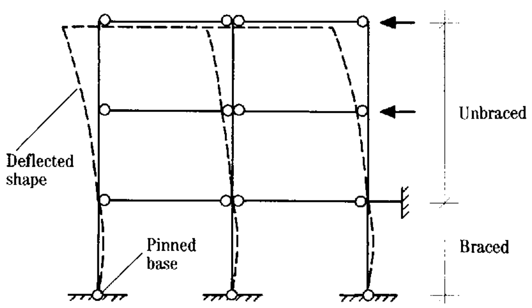
6:1 Options for precast frame stability



a) Braced frame



b) Unbraced frame



c) Partially braced frame

The main advantages of using this approach are given below.

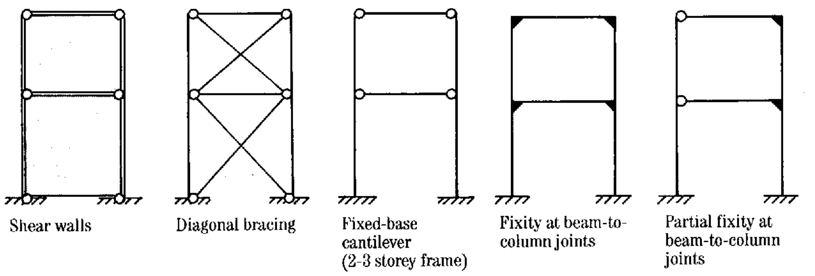
- The minimum number and simplification of precast concrete components. For example, columns may be multi-storey and beams need be provided primarily only in one direction (see Section 3).
- Pin-joined connections can be made without the need for the additional time required to cure in-situ concrete joints.
- The frame is structurally stable, floor by floor.
- The foundations are optimized because the column base is pin-jointed.

The main disadvantage is that shear walls or similar members are often needed in places other than at stairs and lifts, in order to avoid torsional effects. Conflicting architectural and structural requirements occasionally lead to the need for additional framing to produce a satisfactory design.

The secondary effects of changes in length, such as those caused by creep, shrinkage, and relaxation in prestressing tendons, are mostly eliminated by precasting. The practice in design offices is to consider these secondary effects to be taken up in individual joints between the precast components.

With the exception of structural toppings, the use of large quantities of in-situ concrete is confined to places where shrinkage cracking and creep strains can be tolerated and allowed for in design. Where this is not acceptable, non-shrinkable concrete or grout is used.

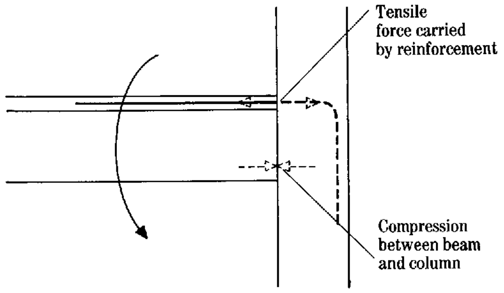
6:2 Provision of lateral stability for precast frames



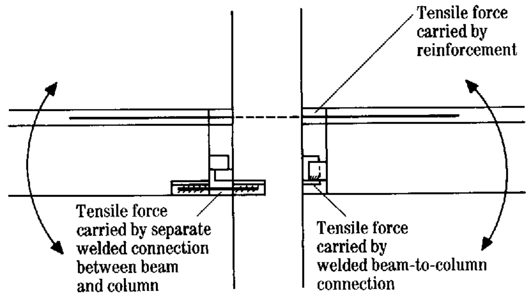
Reinforced and prestressed precast concrete components, and in-situ concrete joints, are designed for strength, frame stability, robustness, fire resistance and durability, in accordance with BS 8110, and other relevant standards. The usual codes of practice for materials, workmanship and safety, are adhered to as necessary. However many of the principles adopted in design, as described in this publication, are not necessarily dependent on codes.

With some items, such as beam-to-beam and beam-to-column connections, designs are based on well established techniques, practical experience and, in many cases, on test results. FIP Recommendations,[19,24,25] the ISE manual,[26] PCI manuals[20,27] and other selected literature,[28,29] are often used to ensure that a common approach, if not identical in detail, is made to design. Structural steel components, and welded and bolted connections such as column base plates, are generally designed in accordance with the appropriate sections of BS 5950 Part 1[30] and BS 4360.[31]

6:3 Provision of fixity at beam/column joint



a) Fixity in one direction



b) Fixity in both directions

DESIGN OF PRECAST CONCRETE COMPONENTS AND CONNECTIONS

Floors

Load-span data for different types of prestressed precast concrete floor systems are presented in graphical form in Figures 6.4a to d. These are based either on ultimate or serviceability moments of resistance, or on ultimate shear resistance, and show typical load v. clear span curves. Maximum spans are controlled by deflection. The actual curves will vary from manufacturer to manufacturer, resulting from differences in profiles, prestress, concrete mixes, fire resistances, etc. The manufacturer's literature should be referred to for precise values.

The data given in Figure 6.4 does not apply where there are substantial line loads at right angles to the span. Handling restrictions may control the maximum length of some units. The design of precast floor units, as distinct from the design of floor systems used in a precast frame, is well documented[19,21] and it is only necessary to deal here with some of the important points relating to their use in precast concrete frames.

The procedure used in the design of precast concrete floor slabs may be carried out in the reverse order from that used in traditional methods. For example, with hollow-core units, a set of pre-determined depths, cross-sections and prestressing patterns is used to compute the limit state strength and stiffness (deflections). Lower bound values are compared with loadings and support conditions. For convenience, the loading is often considered to be uniformly distributed, even allowing for line loads from partitions, and the ends of beams to be simply supported. This gives rise to the direct application of the load v. clear span data shown in Figure 6.4.

Manufacturers can, however, design for individual uniform, point and line loads. In addition, loading conditions such as heavy line loads, point loads, or the concentration of loads around voids, may be dealt with by considering these loads to be distributed onto adjacent units. The general approach, and that given in BS 8110, is to distribute the loads over an effective width equal to the total width of three precast units, or one quarter of the span on either side of the loaded area. This is a simplistic approach and further information on load distribution can often be supplied by the manufacturers, or obtained from other references.[19,24]

In dealing with point loads, BS 8110 calls for test results to justify exceeding the quarter-span criterion which is usually used in design. Depending on their magnitudes and initial bearing areas, point loads may have to be supported on substantial spreader plates.

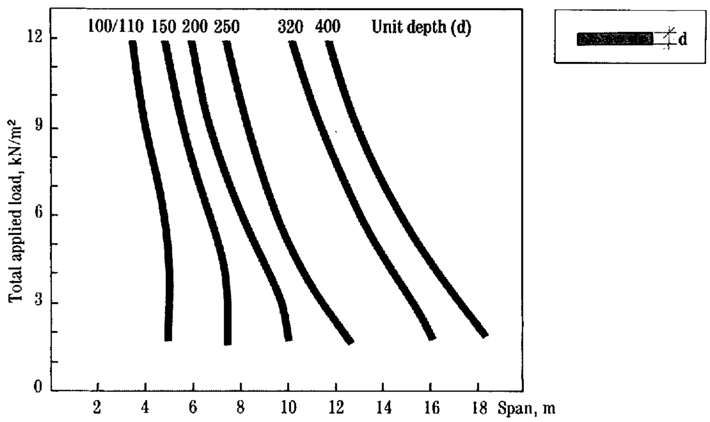
In general, hollow-core units have no reinforcement other than longitudinal prestressing tendons (strand or wire) anchored by bond. These tendons are exposed at the ends of the units, which may be Class 1, 2 or 3 for the serviceability limit state of cracking in accordance with BS 8110. The characteristic 28-day cube strength of these units is at least 50 N/mm^2.

Precamber deflections, which depend mainly on span, prestressing force, eccentricity and quality control, are usually greater than values obtained using an elastic analysis based on theoretical short-term material properties. However, the camber is usually less than the limits specified in BS 8110 and does not exceed span/300 in standard units up to 7 m long. For greater spans, the actual values should be obtained from the design engineers.

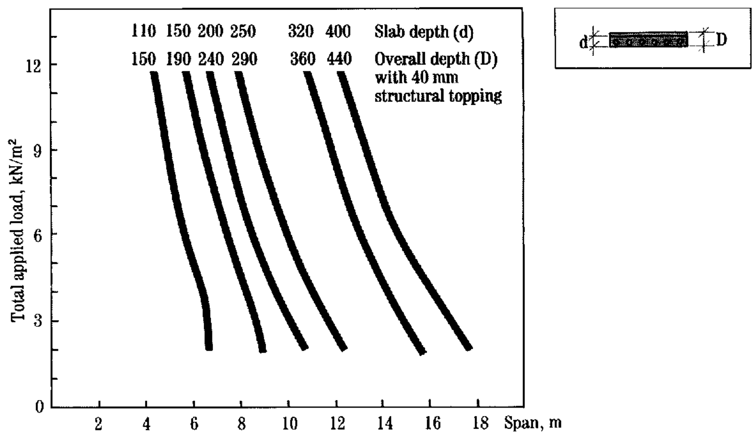
Double-tee units are prestressed longitudinally, both for strength in flexure and in shear, and to control deflections. When required, shear reinforcement is also placed in the webs in anchorage zones. The flanges are reinforced using welded fabric, usually A193, to control shrinkage cracking and to ensure the horizontal distribution of loading to the webs. The units are generally designed as Class 2 members for flexural cracking, although other classes of member can be produced. As with hollow-core units, the characteristic 28-day cube strength of the concrete is at least 50 N/mm^2.

Solid-plank floor units are prestressed or reinforced. Where used, lattice girders are manufactured from mild steel bar, spot welded

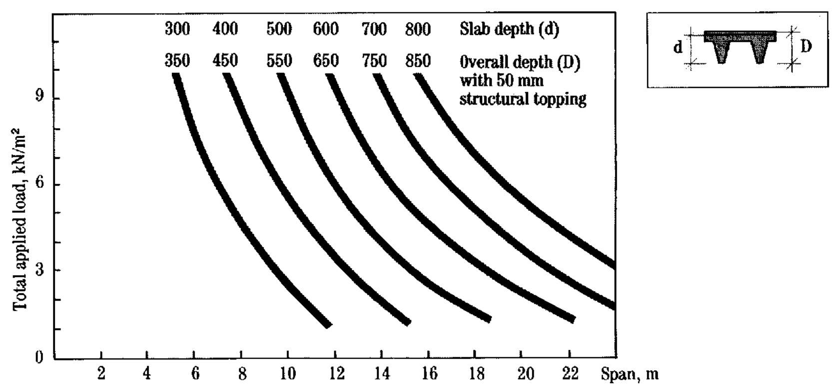
6:4 Load span charts



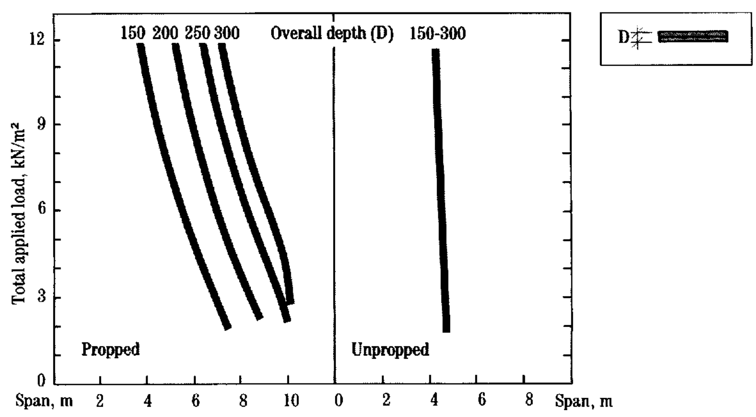
a) Hollow-core units



b) Composite hollow-core units



c) Double-tee units



d) Composite solid-plank units

semi-automatically. The longitudinal bars in the lattices are ignored in design for the serviceability condition, but they may be included in the design for the ultimate limit state.

In most cases, non-structural finishes, including raised timber floors, may be applied directly onto the precast units. Allowances for precamber should be made in calculating the overall floor depth, because the finishes will be thicker at the supports than at mid-span. In-situ reinforced concrete toppings will be required only where there are very large line or point loads, or where the dynamic or acoustic characteristics of the precast concrete floor are considered to be inadequate. Double-tee units, with floor flanges 75 mm thick or more, may have sufficient strength to be used without a structural topping, but they are generally designed, as are flat-plank units, to be used with a structural topping. In some instances hollow-core units are also produced for use with a structural topping to provide composite action. A precast concrete unit with a structural topping produces a composite construction.

Composite floors In composite floor construction, precast units and an in-situ topping form a monolithic floor slab. The design should consider the ultimate strength of the composite floor, the stresses in the precast unit during the various stages of manufacture and construction, and the horizontal shear stresses at the interface between the precast and in-situ concretes. Structural toppings will increase the ultimate moment and load capacity, but the additional dead weight means that they are generally only beneficial in spans up to about 8 m to 10 m. For providing strength, the optimum thickness of topping is about 65 mm at mid-span.

A 40 mm thick structural topping will increase the superimposed load capacity of hollow-core slabs by 20% to 40%, and a 100 mm thick topping will increase it by 60% to 110%. However, when these increased loadings are compared with those for non-composite floors, the efficiency of the flooring, expressed in terms of the volume of concrete used, is reduced by about 8%.

Permissible service stresses are checked at two stages of loading – before and after hardening of the in-situ topping. Different material properties, differential shrinkage and interface bond should all be allowed for in the design. Horizontal shear forces are only likely to be significant for rare loading conditions, such as heavy point loads. In most cases the bond between the in-situ and precast concrete surfaces will be adequate and there will be no need for projecting links.[32] Depending on the design requirements, structural toppings may be reinforced with square mesh, typically A142 or A193, or with nominal lateral reinforcement. The reinforcement may be lapped with projecting peripheral reinforcement in the edge beams and continuous over internal beams. The structural concrete topping should have a minimum thickness of 40 mm, and a strength to comply with the design specification. This should be a minimum 25 grade, although it is more likely to be a 30 grade, particularly for toppings over double-tee units.

Longitudinal joints between floor units Standard edge profiles have been evolved to ensure an adequate transfer of horizontal and vertical shear stresses and to prevent relative displacements between adjacent units. With hollow-core units, the joints are dampened and filled with 25 or 30 grade in-situ structural concrete. Typical joint profiles are shown in Section 4, Figure 4.12. The sides of the units have a natural roughness, caused by the concrete dragging during manufacture, with indentations up to 2 mm deep. The vertical shear capacity is based on a single castellated joint.

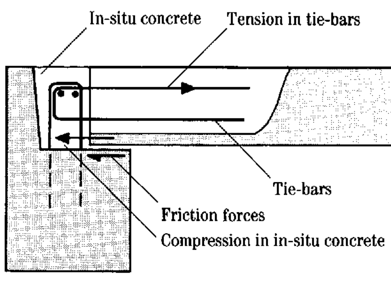
A typical 6 m long x 200 mm deep hollow-core unit may have an ultimate vertical shear capacity of about 275 kN, which is rarely critical. Although generally ignored in design, welded connections between double-tee units, (Section 4, Figure 4.13), provide an ultimate shear resistance of about 25 kN/m in addition to that obtained from the structural topping. The weld is formed between fully-anchored mild steel plates. Stainless steel plates and electrodes may be specified for special circumstances.

Floor-to-beam connections

There are two main categories of floor connections; those at supported joints and those at non-supported longitudinal joints between the floor units and the beams or walls spanning parallel to the units. Both these joints are essential for the integrity of the precast frame.

Connections at supports Although the ends of floor units are normally designed on the assumption that they are simply supported, the couple generated by the contact plane and tie-back (reinforced concrete or weld) (Figure 6.5) may create a degree of end restraint. However, the magnitude of this restraint does not usually affect the flexural behaviour of the floor.[19] The object of the design is to ensure the transfer of vertical loading from the floor to the beam, for both normal

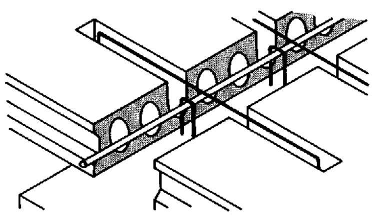
6:5 End restraint in precast slab-to-beam connection



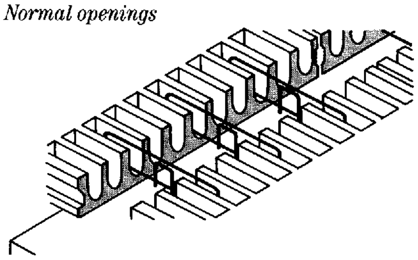
loading and abnormal loading, such as fire or accident. The connection must therefore satisfy the requirements of load transfer, structural integrity and ductility.

During the manufacture of hollow-core floor units, openings are made in the top flanges of the units (Figure 6.6), so that reinforcement and structural concrete, normally 25 to 30 grade, can be placed on site to provide stability or to develop composite action with the beam. Continuity may be obtained, for example, either by directly anchoring reinforcement between the precast beam and in-situ strips, or by dowel action between reinforcing loops, or between loops and other bars (Figure 6.7a). Continuity over internal beams is obtained in a similar way.

■ 6:6 Openings in floor units for site connections



a) Normal openings



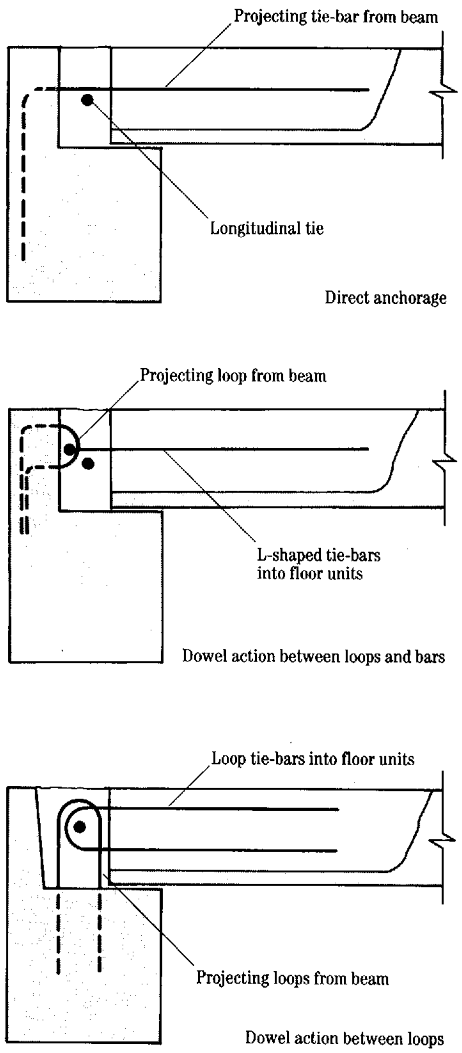
b) Multiple openings

The embedded length of the reinforcing bars is taken as the greater of one anchorage bond length, or that length of reinforcing bar equivalent to the transfer length of the prestressing force in the precast unit. End hooks may be used. The flange openings are designed to be wide enough to ensure good compaction of the in-situ concrete using a small diameter poker vibrator. The infill concrete usually penetrates into the hollow core at the end of the unit for a distance greater than is required structurally.

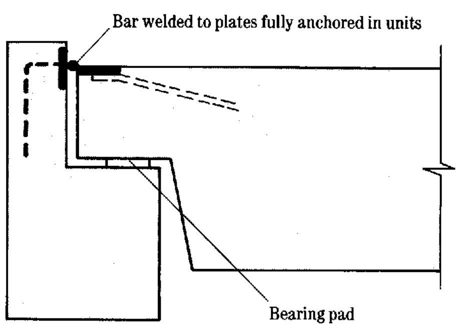
Hollow-core units are usually laid directly onto the floor seatings of the precast beams. In assessing the bearing stress, tolerances must be taken into account, and an allowance made for spalling, etc. A nominal bearing length of 75 mm gives a net length of 60 mm after allowing for spalling, but the design ultimate bearing stress is rarely critical. Rigid neoprene strips or wet mortar bedding have been used under hollow-core units in special circumstances, such as when refurbishing buildings where bearing surfaces are uneven, but they are not structurally necessary in most cases.

In double-tee floor construction, steel plates are provided at intervals in the ends of the precast units and these are welded on site (Figure 6.7b). Anchor bars lap with the main reinforcement as necessary. A minimum nominal bearing length of 150 mm is recommended, and the units should always be seated on pads of neoprene, or similar material, with a minimum area of 100 mm square.

■ 6:7 Beam-to-slab continuity



a) Hollow-core units



b) Double-tee units

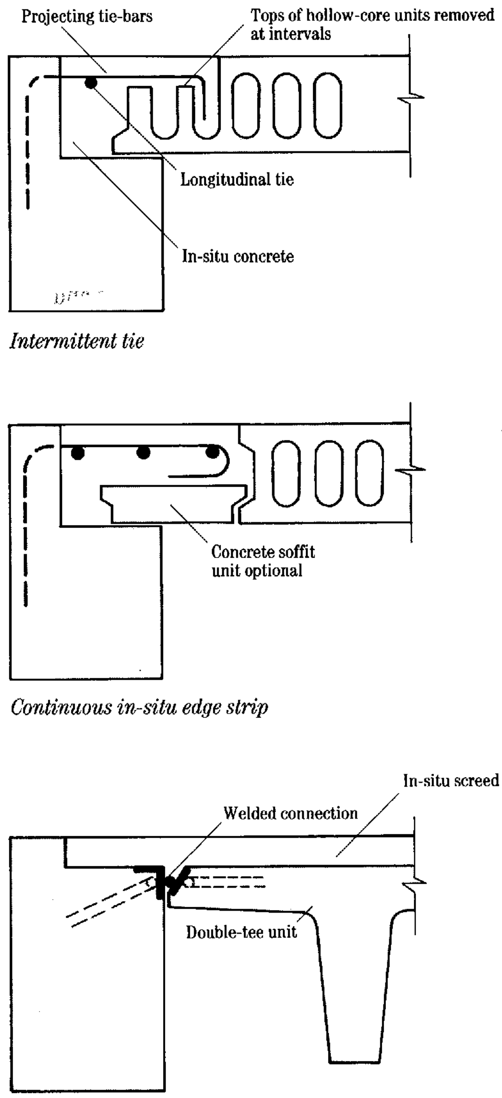
Connections between flat-plank flooring and supporting members present few problems, but a cement-sand mortar bedding may be required, particularly where 2.4 m wide units bear onto precast beams, and especially where the beams are prestressed and have a camber. A nominal bearing of 75 mm is normally sufficient for flat planks. Continuity of reinforcement can be provided by lapping the mesh with reinforcement projecting from the beams or walls.

Connections at longitudinal joints The main function of the longitudinal joints between the edges of precast floor units and beams or walls running parallel to the floor, is to transfer horizontal shear between the floor and beam, whilst allowing for the differential movement between adjacent precast components.

Recesses for connections may be formed in hollow-core floor units by removing part of the top flange. In-situ reinforced concrete joints are cast intermittently along the edge of the slab, usually at 2.4 m centres, or continuous in-situ joints may be used where a stronger shear key is required (Figures 6.8a and b). These may be formed either by using a soffit unit, or may extend to the full depth. The slab is sufficiently flexible to accommodate differential vertical movements caused by temperature fluctuations and loads.

Typical longitudinal joints between double-tees and edge beams are shown in Figure 6.8c. Here welded connections are made at 2.4 m intervals immediately after fixing. The strength of the weld is sufficient to maintain the double-tee units in position whilst the in-situ concrete topping is hardening. The welded connection is designed to prevent the topping, which is tied to the edge beam, from separating from the flanges of the precast concrete double-tee units. The flanges of the double-tee units are reinforced with mesh to cater for flexural forces generated by the shear restraint of the weld caused by differential movement.

6:8 Connections at longitudinal joints



a) *Intermittent tie*

b) *Continuous in-situ edge strip*

c) *Welded tie to double-tee*

Floor connections at loadbearing walls or cladding panels

The primary forces in these connections arise from vertical compression from upper-storey panels, or horizontal shear from floor plate diaphragm effects.

By comparison, secondary forces caused by temperature fluctuations, long-term shrinkage and creep, and bending moments induced by end restraints, are of minor importance.

Connections at wall supports require careful detailing, particularly if the floor units are supported within the thickness of the walls and large wall loads are imposed. The possibility of web crushing may require some hollow-core units to be strengthened by infilling their voids to a depth coincident with the edges of the walls (Figure 6.9a). It may be possible to support light loads directly onto unfilled hollow-core units (Figure 6.9b). Various details are used for the external joints, one of which is shown in Figure 6.9c.

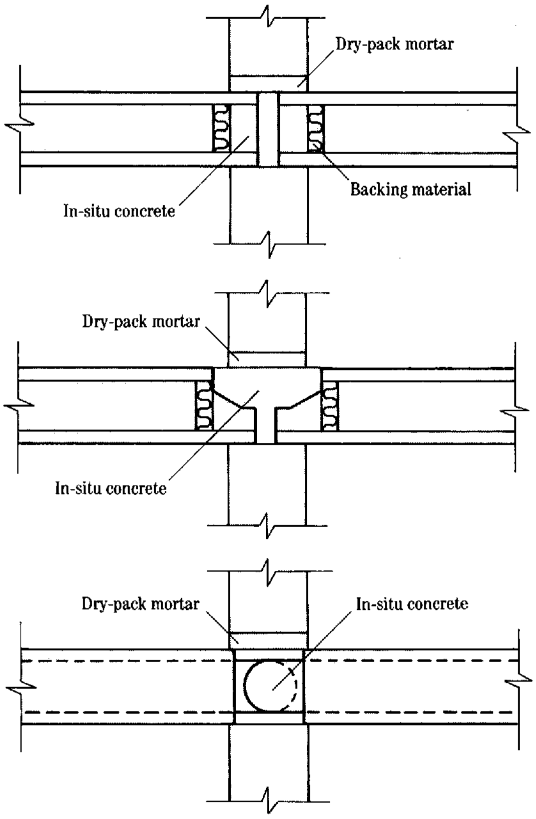
Double-tee units may require rib-end closure pieces, or they may be supported in specially shaped recesses (Figure 6.9d).

The main factors controlling the transfer of vertical loads are: the extent and compressive strength of confined in-situ concrete; the effective width of shims and the use of steel levelling shims in the upper-storey panel bearing; the vertical splitting strength of the panel ends; and the strength of any mechanical connection.

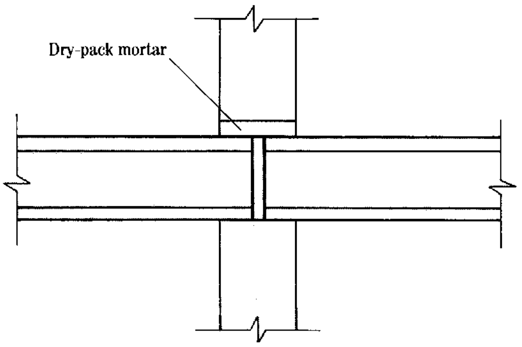
Design of main (support) beams

Reinforced or prestressed concrete beams in a precast concrete frame are designed, using a straightforward analysis, for the specified loadings and support conditions. The design may be complicated by

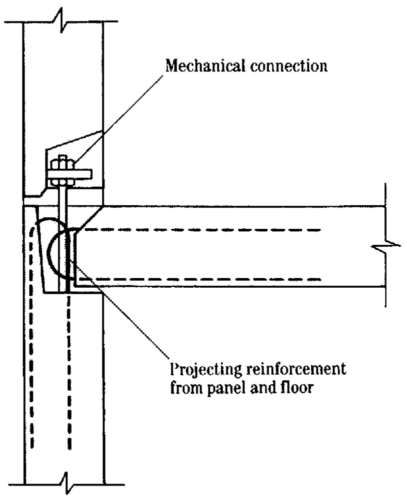
6:9 Connections at loadbearing walls



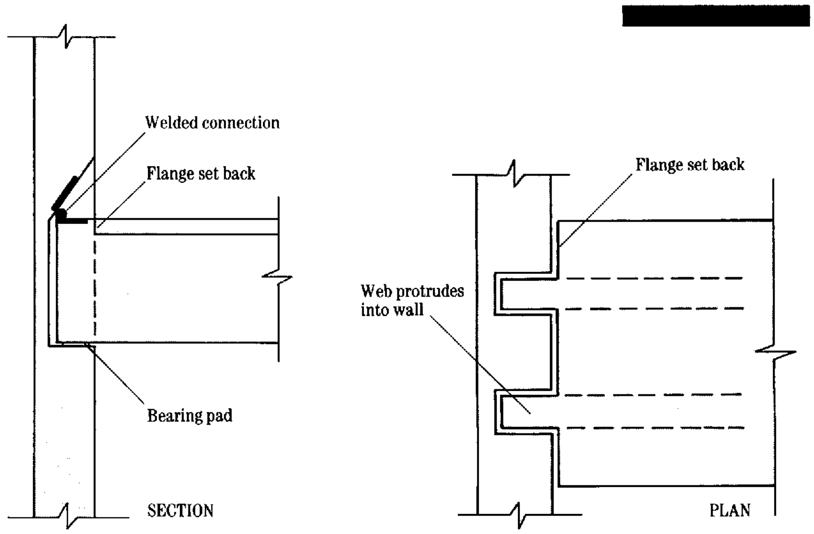
a) Heavy loads: hollow-core units



b) Light loads: hollow-core units



c) External wall-to-floor joint



d) Double-tee units

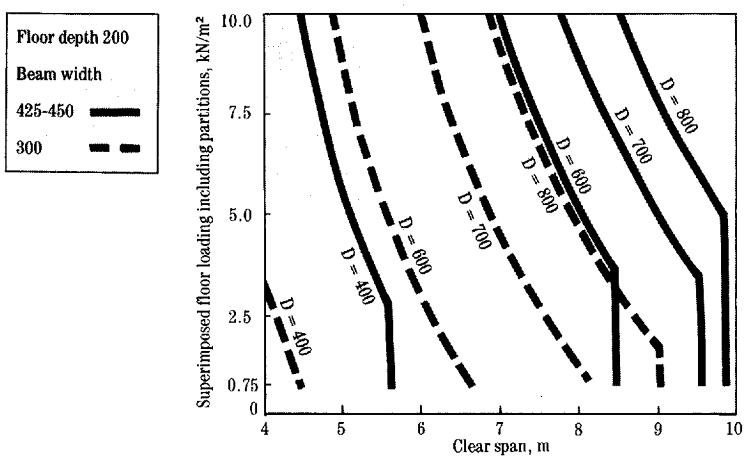
beam-to-beam connections, large asymmetrical loadings or the provision of service holes near the ends of beams, where special shear cages or shear boxes are provided to transmit shear forces to the support. The design may be either non-composite, in which the in-situ concrete infill between the ends of the slab and beam is ignored, or composite, where the infill and, possibly, the floor unit act with the beam.

Non-composite beams Standardized designs may be prepared for beams with cross-sections of similar shape, but which vary in depth, width and quantity of reinforcement. Ultimate moments and shear resistances are thus equated to particular project requirements. Load v. span data may be presented for a range of components: Figures 6.10 and 6.11 are based on 6m span floors. Families of curves or tabulated data are usually prepared in advance. Standard calculations often allow for service holes, up to 50 mm in diameter, with their axes located near to the neutral axis of the beam.

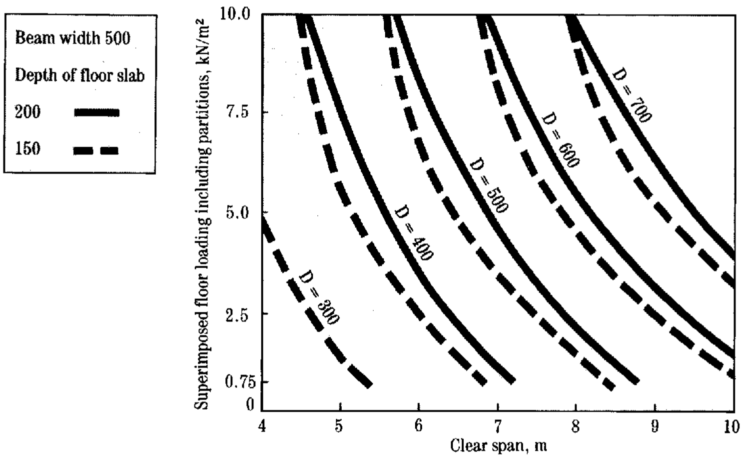
Characteristic concrete strengths for reinforced and prestressed beams are typically 40 N/mm^2 and 60 N/mm^2 respectively. Reinforcement, including designed links in reinforced precast concrete beams, is normally high-tensile hot-rolled deformed bar. Prestressed concrete beams are reinforced longitudinally with prestressing strand.

The design of reinforced concrete beams is based on conventional analysis. A wide range of preferred cross-sections is available, and designs with non-preferred sections are easily accommodated. Edge beams subjected to asymmetrical loading are not always reinforced against torsion because the floor plate provides a horizontal prop force and the lateral stiffness of the beam is sufficiently large to prevent excessive horizontal deflections under the action of the propping force. The minimum requirement is that an ultimate horizontal tie force of at least 40 kN/m must be provided between the floor slab and the edge beam.[33] Edge beams with narrow upstands are checked for lateral stability or slenderness in the temporary condition before the in-situ concrete connection to the floor is made. The slenderness of the upstand may restrict the span. Rectangular, L- and inverted-tee beams may be doubly reinforced in flexure.

6:10 Load v. span – edge beams



6:11 Load v. span – internal inverted spine beams



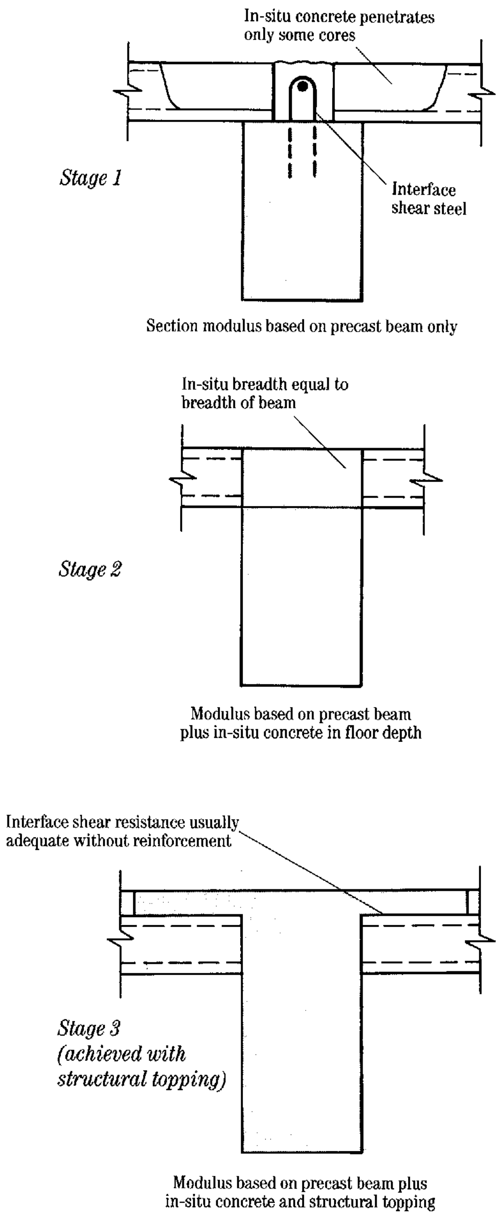
Special attention is given to the shear reinforcement near to end connections. Links and bent-up reinforcing bars are provided to ensure the transfer of shear in the critical region. In some instances the reinforcement cage is partially or wholly replaced by a prefabricated shear box made from welded angles or rolled steel hollow sections. These boxes are used to:

- Transfer the shear forces to a point in the beam where the links are considered to be fully effective.
- Prevent bursting of the concrete at the ends of the beam.
- Provide a steel bearing plate to make a steel-to-steel connection to the supporting member; extensive testing has been carried out to support the design methods used and to prove the satisfactory performance of both the connections and units.[33]

The design of pre-tensioned prestressed concrete beams is based on a pattern of tendons predetermined by the arrangement of holes in the jacking heads. Although beams are generally limited to symmetrical sections of modular depth – usually in 50, 75 or 100 mm increments – considerable architectural freedom is still possible. The load capacity is nearly always governed by the prestressing forces at the serviceability limit state. Class 2 members are usually specified. Unbonded and deflected tendons have been used in special cases, but complications in the long-line prestressing system tend to outweigh the improvements in structural performance.

Composite beams As with composite floors, service and ultimate stresses in composite beams are checked at two stages of loading, or at three if a structural topping is added to the floor. The essential features of the design procedure are outlined in Figure 6.12.

6:12 Composite action in precast beams



Stage I stresses occur only in the precast beam. These are the result of prestress and relaxation (if the beam is prestressed), creep before the in-situ concrete has hardened, and the self-weight of the beam, slab and wet concrete. A notional allowance is made for construction traffic. If there is an extensive amount of in-situ concrete, this will be about 1.5 kN/m^2.

Stage 2 (and 3) stresses result from superimposed loads, services and partition loadings, differential shrinkage and the total creep relaxation, after hardening, of all the in-situ concrete. These are added to the Stage I stresses to give the stress in the composite beam.

Ultimate moment and shear resistances are computed in the usual manner. Differential shrinkage and creep strains are ignored at the ultimate limit state. Interface shear requirements, such as roughened surfaces and interface reinforcement, are carefully allowed for when detailing precast concrete components.

An important question which may require clarification, is what to assume as the effective width of the compression flange of a composite beam. This is particularly so if the cores in certain types of hollow-core floor units are filled only intermittently (Figure 6.6a), which is the common practice with slipformed units.

Here the minimum section is through the unfilled hollow core. This consists only of the top and bottom flanges of the slab – each usually 20 mm to 30 mm thick. Although interface shear transfer between beam and slab may only be fully effective at the positions of the intermittent reinforced in-situ filled cores, representing possibly 1/12 of the span, there is experimental evidence to suggest that sufficient horizontal shear stresses can exist to generate a degree of composite action. In such cases the effective width of the compressive flange may be at least equal to the width of the supporting precast beam. It is possible that further information may be obtained from the experimental work carried out by various manufacturers, both directly and through universities.

In the unlikely case where every core is opened up, tied to the beam and filled with in-situ concrete, the effective width of the flange (based on the full depth of the in-situ concrete) may be taken as at least equal to the distance between the extremities of the opened core. Openings in cores are usually about 600 mm in length, resulting in effective widths of about 1350 mm and 700 mm in symmetrical and non-symmetrical arrangements, respectively.

The effective width for the in-situ concrete topping used with solid plank flooring is assumed to be the same as for monolithic reinforced construction.

Beams are not designed to act compositely with double-tee flooring units.

The fire resistance of beams is based on the recommendations in BS 8110. Concrete beams in the preferred range shown in Figures 4.6 and 4.7, are typically designed to have a fire resistance of 2 hours, although 4 hours may be obtained without additional external protection. Most prestressed concrete beams have a fire rating of 1½ to 2 hours, but 4 hours may be obtained by increasing the average cover to the prestressing tendons, or by other means.

Beam-to-column and beam-to-wall connections
Perhaps the most fundamental feature in the design of multi-storey precast concrete frames is the connection between vertical and horizontal components. There are two broad sub-divisions:

- Where the vertical member is continuous, both in design and construction, and horizontal components are framed into it at various levels.
- Where the vertical member is discontinuous, but only in construction, and the horizontal components are either structurally continuous or structurally discontinuous across the joint.

Although the first sub-division usually refers to skeletal column and beam construction and the second to wall frames, there is an alternative approach, particularly for low-rise unbraced buildings, in which single-storey-high columns are erected floor by floor with, in some cases, special reinforced in-situ concrete connections between the precast components. The joint design is typically based on truss, corbel or half-joint theory, often initially justified by tests.

Connections to continuous columns using steel inserts These joints are the commonest form of beam-to-column connection used in the UK. The structural mechanism depends on static strength, stiffness, load transfer into the connecting elements, temporary stability and structural integrity (in both direction and magnitude) at loads in excess of design values. A wide range of joints has evolved to satisfy these requirements.

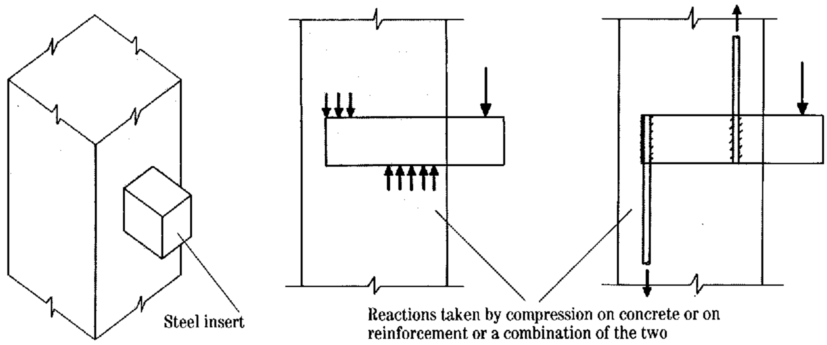
The joint is designed to:

- Transfer the shear force carried by the reinforced concrete section at the ends of the beams into the connection by using vertical shear links and/or bent up bars in a prefabricated steel section, called a shear box.
- Ensure adequate shear capacity in the plane of the physical discontinuity between the beam and column (or other supporting member), taking into consideration static strength, cracking, durability and fire resistance.
- Transfer compressive loads into the reinforced concrete column. (Figure 6.13).

Horizontal bursting forces are resisted by closely spaced horizontal links or U bars. Column connectors are generally either fully-anchored cast-in-sockets, steel boxes or billets, or H-section inserts.

6

■ 6:13 Column insert design



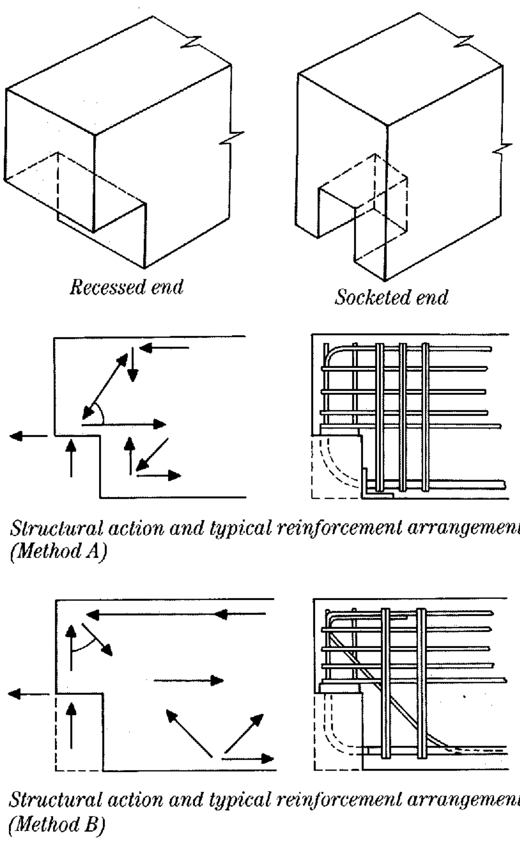
Beam end design Recessed and socketed beam ends (Figure 6.14) may be designed in one of two ways, using alternative truss analogies, depending on the depth of the recess and the depth of the beam. In both cases sufficient reinforcement is provided, together with mechanical or physical anchorage, to enable truss action to develop in the beam. The resulting factors of safety against ultimate loads are usually large.

In the first method (Figure 6.14, Method A), the end of the beam is reinforced with an arrangement of vertical links and horizontal bars to cater for tie forces and bursting pressures in the compression zone. The horizontal plates in direct contact with the bearing surface are used to avoid the risk of local spalling and crushing, and to ensure uniform bearing pressures in this highly stressed zone.

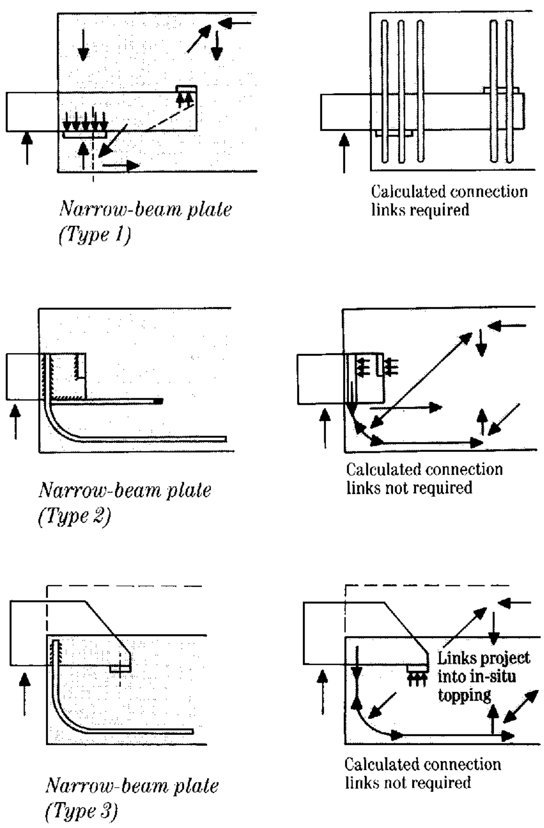
In the second method (Figure 6.14, Method B), reinforcement normal to the potential cracking plane provides the force required to maintain the ultimate shear resistance by shear friction. The diagonal bars are fully anchored into the top compressive strut and are positioned to prevent diagonal cracking in the critical shear plane at the corners of the pocket. Horizontal hairpin bars are placed directly over the top of the pocket as a restraint against lateral bursting forces. The bars are held in position by an equal amount of fully-anchored vertical steel. They form the basis of the shear reinforcement cage, in which the spacing of the links is gradually increased as in normal reinforced concrete design.

Alternatives to the shear cage concept outlined above, are the narrow beam plate (Figure 6.15), and the shear box, in which a solid plate, or other structural steel section, projects from the end of the beam. Designs are based on ultimate bearing stresses in both the plate and the concrete beam, the prevention of spalling, bursting and splitting, and on an adequate tie-back in the concrete beam. The ultimate shear capacity of the section is based on the shear capacity of the shear box itself. The shear force is gradually transferred into the reinforced concrete beam. Tie-back forces are distributed into the concrete beam, either by a concentration of vertical stirrups, or by welding a wide steel plate (or similar) to the bottom of the shear box.

■ 6:14 Recessed and socketed beam ends



■ 6:15 Beam connector – projecting plates



Factors of safety for the shear capacities of precast beams using shear boxes, or shear cages and bearing plates, are usually obtained by individual manufacturers from full-scale laboratory testing. The results show that at ultimate conditions, despite local cracking close to the bearing plate, ultimate failure is a function of the shear reinforcement, just as in ordinary reinforced concrete. Shear cages and boxes are usually rated in increments of 50 kN up to 300 kN, and at 100 or 150 kN increments thereafter. The practical limit, which varies with different design and manufacturing techniques, is about 750 kN. Further design information is given in References 20 and 26.

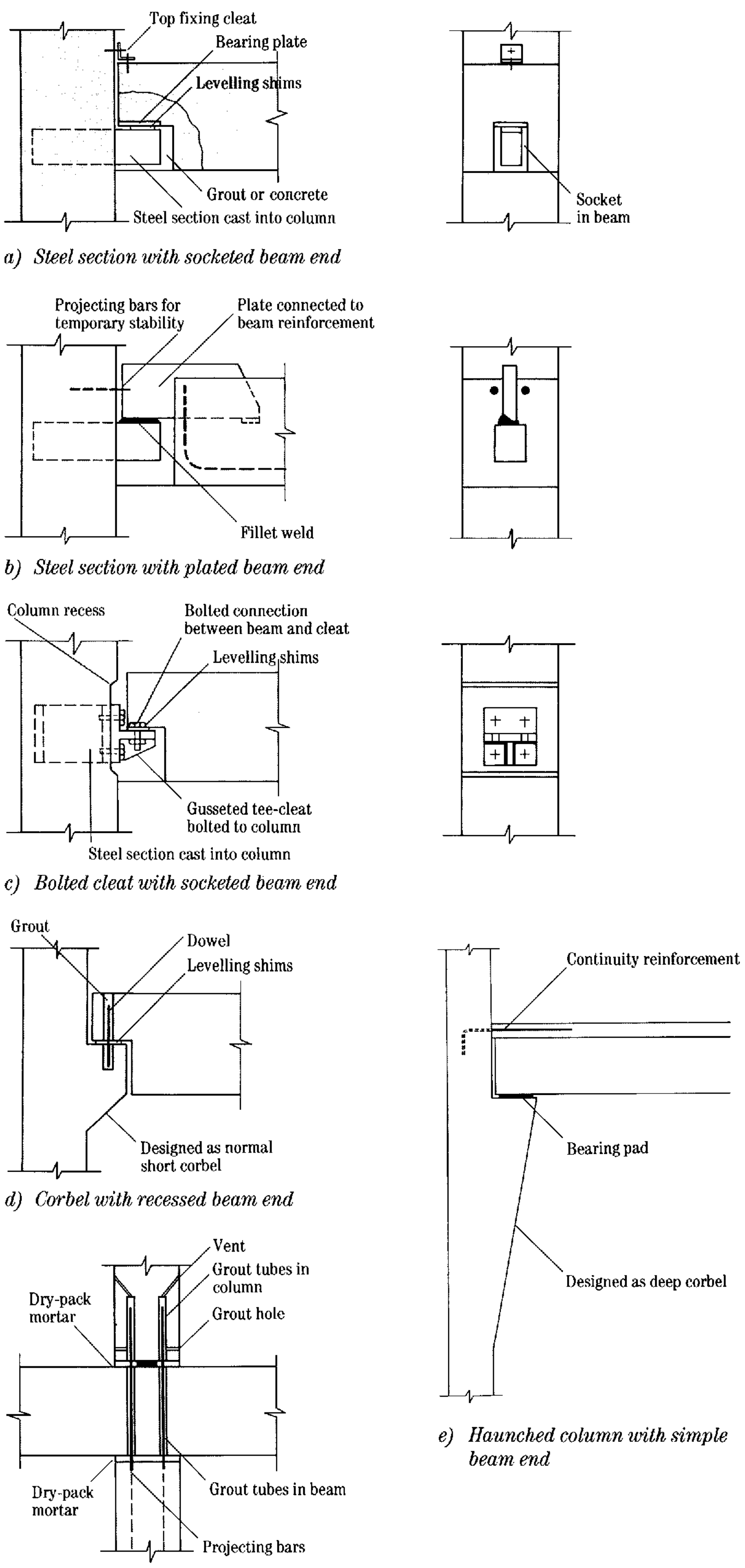
Beam-to-column connections Various types and combinations of beam-to-column connections can be produced. The types commonly used are shown in Figure 6.16 and described below.

- A direct bearing between beam and column inserts. Here there is no positive mechanical connection between the precast concrete components (Figure 6.16a). A top fixing is provided. This can be either a bolted or welded cleat or plate, or an arrangement of reinforcement, but it is ignored when calculating the shear capacity. This detail may also include a drop-bolt located between the top fixing and the column insert.
- A welded connection (Figure 6.16b). In this the top fixings may be omitted at the erection stage because of the stability provided by the weld. A variation of this is to use an inverted-T steel section as the column connection. This gives a lower weld position, which increases the joint's moment capacity, but its use may be limited by its shear capacity.
- A separate intermediate cleat (Figure 6.16c). This may be a rolled structural tee, a rolled angle, a fabricated plate angle, or a fabricated box. The top fixings may be omitted because of the stability provided by the bolts. The shear capacity is based on the strength of the bolts and gusseted cleat.

Beam-to-column connections are designed for the least favourable positions of the contact surfaces, taking into account the accumulation of frame and component tolerance which may, in severe cases, approach 15 mm. The connection detail is often designed so that only a small gap remains between the beam and column. This gap is filled using a cement-sand, or neat cement grout, which may contain a proprietary expanding agent.

In some instances, particularly where the cover to the surface of the nearest steel insert exceeds about 50 mm, a small cage is formed by spot welding, or otherwise attaching small diameter links to the inserts. The grout normally has a minimum design strength of 30 N/mm^2 and provides fire protection and durability to the steel connection. A fine aggregate concrete mix may be used for larger gaps. In addition to supporting the main beam, the design and detailing of the connection needs to allow for any torsional moments that may arise during service and erection (see *On-site connections* in Section 7 page 83).

6:16 Beam-to-column connections



a) Steel section with socketed beam end

b) Steel section with plated beam end

c) Bolted cleat with socketed beam end

d) Corbel with recessed beam end

e) Haunched column with simple beam end

f) Continuous beam with discontinuous columns

Column insert design Column inserts supporting precast concrete beams may be either solid inserts (Figures 6.16a and 6.16b), or cast-in thin-walled rolled sections (Figure 6.16c). This part of the joint has been the subject of considerable research and analysis,[34,35,36] and may be designed with as much confidence as the other joints in the precast frame.

The design of solid inserts varies between wide sections, where the width of the bearing surface is from 75 mm to 0.4h, and thin plates, which include thin-walled rolled sections with a wall thickness less than 0.1h, where h is the width of the column.

For concentric loading, the ultimate capacity for a 300 mm square column with a 100 mm wide insert is about 1000 kN, so it is rarely critical. The column connector is given a load rating according to the design and ultimate load, tested under eccentric loading. The ISE manual[26] proposes a method for determining the load and moment capacity of prismatic sections which is adopted by many design engineers.

The equilibrium of forces and moments at the ultimate limit state is achieved by reinforcing, or otherwise guarding against bursting, spalling and splitting. The general practice is to use closely spaced links to guarantee the confinement of concrete directly above and below the column insert, and to ensure that the insert does not interrupt the main longitudinal reinforcement. Thin plate inserts are nearly always supplemented by tensile anchorage reinforcement, or by a wider bearing plate close to the face of the column.

Reinforcement is welded to the sides of the plate and the compressive bond resistance of these bars is used if there is insufficient anchorage bond length available above the connection, such as occurs at roof level. Links are used, in the usual manner, to prevent lateral buckling of these bars.

With cast-in thin-walled rolled sections used with bolted cleats, the flanges of the insert are recessed about 25 mm from the face of the column to overcome edge spalling and to ensure that the concrete in direct contact with the flange is confined by the column links.

Cast-in sockets are used to transmit horizontal forces to the column and, occasionally, for carrying vertical forces. Anchorage reinforcement is attached to the socket where this is required to take a tensile force. Vertical shear resistance is provided by direct bearing under the barrel of the socket. This type of connection has a maximum shear capacity of about 250 kN.

Connections to columns using corbels Very heavy loads, or long-span beams, are sometimes supported on column corbels (Figure 6.16d), but the use of corbels may be restricted where all connections must be contained within the overall depth of the beam and slab so as to reduce the depth of the structural zone. This may occur, for example, where it is unacceptable for the connection to project below ceilings or into the service zone. Shallow corbels, designed as short cantilevers, may be used where the structural depth is limited. The design procedure is well documented in BS 8110[8] and PCI literature.[20,27]

Some manufacturers' production manuals contain standard column corbel details, but corbels are more often designed individually to suit the specific requirements of the project. The design and detailing is carried out with the usual attention to preventing spalling, cracking, etc. If a dowelled connection is made between the beam and corbel, horizontal hairpins are used to prevent bursting of the thin cover to the dowel hole in both corbel and beam. Site continuity reinforcement may be required to control tension resulting from beam movements. Steel-to-steel connections are made by anchoring a bearing plate into the shoulder of the corbel.

Connections to haunched columns The simplest design is just the provision of an adequate bearing surface at the level of the beam soffit. The method of fixing is similar to that for shallow corbels. Except for its use in car-parks, this type of joint is not widely used in multi-storey frames because of the increased dimension of the column at each floor level. It is more likely to be used in low-rise unbraced structures, where the increased column depth improves cantilever action, or where the magnitude of the beam's end reaction exceeds about 750 kN and cannot be carried practically or structurally by a shallow corbel or steel insert.

A top fixing, similar to that used with steel inserts, is used. This can be a bolted or welded cleat or plate, or continuity reinforcement across the column. The connection is essentially a pinned joint, but it can develop a considerable hogging moment capacity by using extended bearings and reinforced in-situ concrete acting compositely with the precast beam. Figure 6.16e illustrates the basic principles of this.

Connections between beams and discontinuous columns In these connections, the horizontal components are seated on a dry-packed mortar joint on the top of the vertical members. This type of joint is mainly used where beams need to be continuous over the support, such as to form a cantilever. In this situation the projecting reinforcement in the lower column section is passed into the upper column section through sleeves in the beam. It is then fully grouted to provide vertical continuity, as shown in Figure 6.16f. Moment continuity is not generally considered in the design of these joints, but may be provided if required. Examples are shown in the *PCI design handbook*.[20] The joints between beams and columns are often formed by seating the components on neoprene pads or steel shims. If steel shims are used, the joint, which is normally at least 20 mm thick, is filled with a dry-pack mortar.

Another common connection is that at the top of a continuous column, usually at roof level. Here the beam is seated and dowelled onto the column head to form a simple support. There is normally no requirement for steel-to-steel bearings, except where welded continuity splices are necessary.

Design of columns and walls subject to gravity loading

When the questions of manufacture and different types of structural connections have been resolved, the normal principles of design for in-situ reinforced concrete columns and walls are applied. Except for a few isolated cases, precast concrete columns are designed to the recommendations given in BS 8110 for members subjected to combined axial compression and bending. An exception is where columns are axially prestressed to enable very long units to be pitched without flexural cracking. Grade 40 or 50 concrete is usually specified but, because of the early strength required for lifting units in the factory, the actual characteristic strength is often about 70 N/mm^2.

Structural design commences with an assessment of frame stability, and of the axial loadings and bending moments at each floor level. Braced columns in no-sway frames are checked for slenderness. An effective length factor of 1.0 is usually assumed, although this may be reduced if there is adequate analytical or experimental evidence showing the effects of moment rotation stiffnesses in the beam-to-column connection. The base of the column is usually considered to be a pinned connection, even where there may be moment fixity from the type of pocket foundation used extensively to simplify column manufacture.

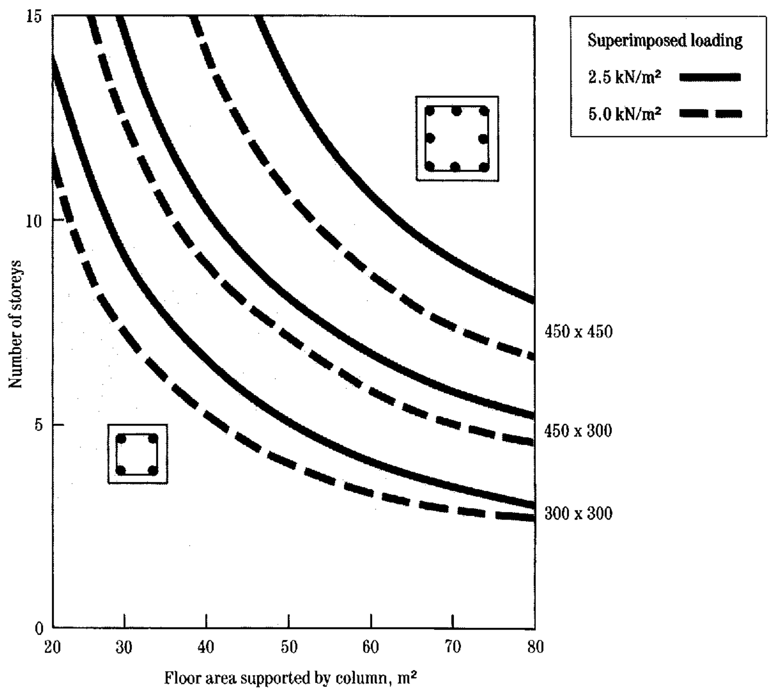
The bending moments in the column at beam level are determined from the eccentric loading in the connection. The eccentricity varies with the type of connection, and typically ranges from h/2 + 65 mm to h/2 + 165 mm, where h is the depth of the column. The resulting bending moments are normally distributed in the column in proportion to the stiffnesses, EI/L, of the column sections on either side of the joint, and the column then designed accordingly. Figure 6.17 gives an indication of the required sizes of short rectangular columns, when used in a braced frame, related to the number of storeys and the floor area. Braced slender columns are analysed in the usual manner, taking into consideration the second-order deflection, or additional bending moments as appropriate.

In sway frames, the horizontal force is shared between the columns in proportion to the flexural stiffnesses of the columns and degree of moment fixity provided at the foundation. Most unbraced columns are slender, although columns acting as 'wind posts' are often deliberately proportioned so that they can be designed as short columns. An effective length factor of 2.2 is normally taken for cantilever columns in pin-joined frames founded on moment-resisting bases.

Additional bending moments from second-order deflections are calculated at every floor level. These moments are based on the total axial load at that level, and the effective height of the column (actual height x 2.2) from the foundation to that level. Bending moments from eccentric loading, horizontal forces and second-order deflections are combined to give the most onerous design condition. The resulting bending moments generally preclude the use of pinned bases and most column splices in sway frames.

In sway frames, the magnitudes of the forces and moments restrict the capacities of most columns to two storeys, particularly if the ground to first floor height exceeds about 3.5 m. This may be extended to three storeys if the load from the roof is very light, such as from steel rafter and sheeting construction.

6:17 Typical column load capacity chart



The fire resistance of precast concrete columns is based on the normal recommendations for reinforced concrete columns given in BS 8110. The minimum rating is two hours for a 300 mm fully exposed face. The maximum rating is four hours. Lightweight aggregate concrete is not often used for columns because of limitations on localized bearing capacities at the beam connection.

Column splices

'Column splice' is the general term for a joint where a horizontal structural connection is made between a column and another precast component. The lower member is usually a column, but it may be a wall, a structural cladding panel, or a beam. It does not include the connection to bases or other foundations; these are dealt with later.

Column splices are often staggered to assist erection and to provide stability during construction (see Section 7). The first splices are usually distributed between the third and fourth floors, except in five-storey frames where the splices, if used at all, are made at second and third floor levels. Splices are located either at a floor level within the structural floor zone where they may be concealed in the floor finishes, or at a convenient working height above floor level, say about one metre, where they are nearer to the point of contraflexure.

Column-to-column splices Continuity of vertical elements is essential for structural stability. Continuity at column-to-column splices is provided, either by coupling, welding or bolting together mechanical connectors anchored into the separate precast components, or by continuity of reinforcement through a grouted joint. Figure 6.18 shows some of the more commonly used splices.

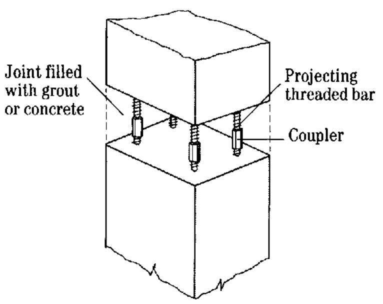
The coupled splice (Figure 6.18a), provides a mechanical tie between the precast components which is capable of axial load and bending moment interaction. The strength of the splice may be controlled by the threaded couplers. The projecting reinforcing bars in the upper and lower columns have threads of opposite hands to enable the couplers to be screwed on. The strength of the in-situ concrete infill must be at least equal to the compressive strength of the concrete in the column. Links may be required in the infilled region to provide stability to the compression reinforcement, and to increase the strength of the infill by confinement. An expanding admixture may be necessary to control shrinkage of the infill concrete. The height of the splice is generally less than 200 mm.

These joints are normally filled in two stages; firstly by filling with concrete to within about 10 to 20 mm of the top of the splice, then by filling the remaining gap at a later date with dry-pack mortar or grout. This is particularly important when dealing with mortar or grout volumes greater than 0.05 m^3 or where the height of the splice exceeds 300 mm.

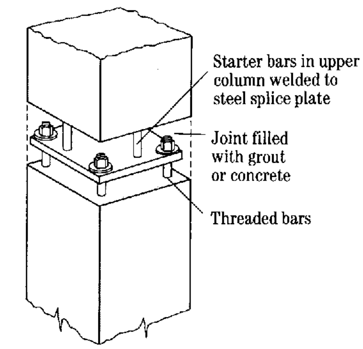
With welded-plate splices (Figure 6.18b), reinforcement projecting from the lower column section is threaded and clamped by four nuts to a steel plate welded to bars projecting from the upper column section. The compressive strength of the splice is based on the strength of the infill concrete plus either the load capacity of the threaded bar and nut system, or the load capacity of the welded mild steel reinforcement. The flexural and shear strength of the plate is increased by being confined by the in-situ concrete.

Welded lap joints (Figure 6.18c), are not widely used because of the difficulties in restraining the reinforcement in the joint, which may need to be

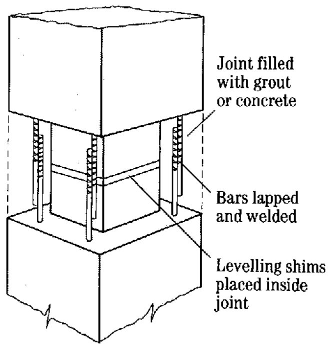
6:18 Column-to-column joints



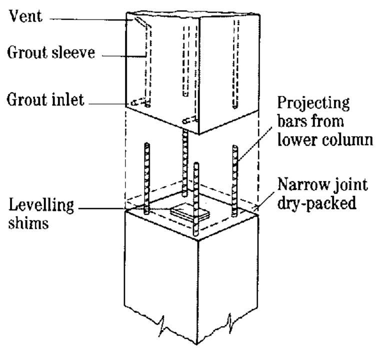
a) Coupled splice



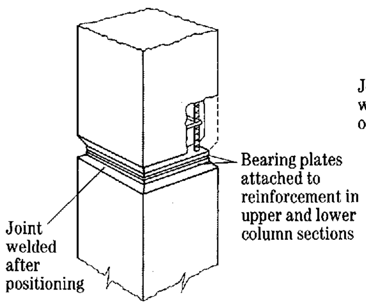
b) Welded-plate splice



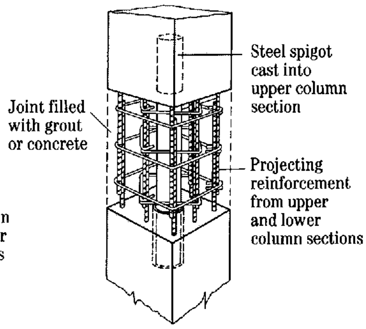
c) Welded lap joint



d) Grouted splice sleeve



e) Welded bearing plate connection



f) Special column connection

500 to 700 mm high to provide the necessary weld length. A single link can be placed in the gap, but this must then be made wider and may be difficult to dry-pack. Nevertheless, there are many variations of this type of joint and it can be suitable in some situations.

Grouted sleeve splices (Figure 6.18d), are the most popular and economical column splices. A full-scale testing programme[37] has shown that the axial load-moment characteristics of this type of splice are equal to those of the parent column. These splices may be made at almost any level in the column, but they must be made carefully.

Welded bearing plate connections (Figure 6.18e), have also been used. In these, a steel plate is accurately fixed at each end of the column sections and anchored back into the column by welded reinforcement or steel sections. This produces close-coupled joints which are directly welded around their perimeters.

Many types of column-to-column splices can be devised, but it is important to ensure that they can be satisfactorily manufactured, and used under site conditions. An example of a special column connection is shown in Figure 6.18f, taken from the *PCI design handbook*.[20]

Columns spliced onto beams or other precast components Pin-jointed splices can be formed between precast columns and beams by using any of the column-to-column splices. In unbraced sway frames, large torsional moments may be induced in the beam and the joint may have to be designed to eliminate moment transfer. A pin-jointed version of the welded-plate splice is shown in Figure 6.19. Other grouted or bolted details can be produced. Moment-resisting splices may be formed between columns and precast walls, because walls are usually capable of accommodating the transferred moment.

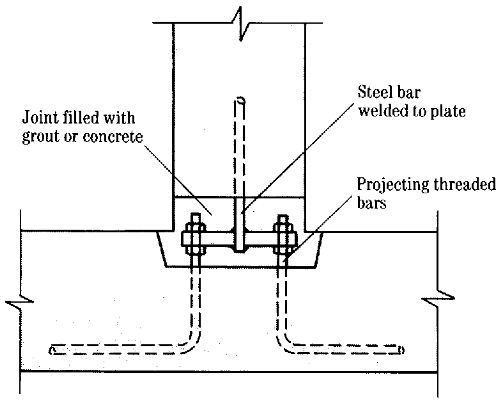
There are occasions when, to satisfy an architectural feature, it may be necessary to support column loads on floor units. Lightly loaded columns, for example those supporting a mansard roof, have been successfully supported on both hollow-core and double-tee floor units. The varying positions of the cores and flanges of the floor units make it necessary to pay particular attention to detail, so as to avoid the transfer of moments.

Column base joints

The design of the connections of columns to pad footings and other in-situ, or precast concrete foundations, is well documented,[20,26] but the important points relating to their design and use are included here. The main methods of connection are:

- Grouted pockets.
- Base plates, with plan dimensions greater or smaller than the dimensions of the column.
- Grouted sleeves.

6:19 Column-to-beam splice

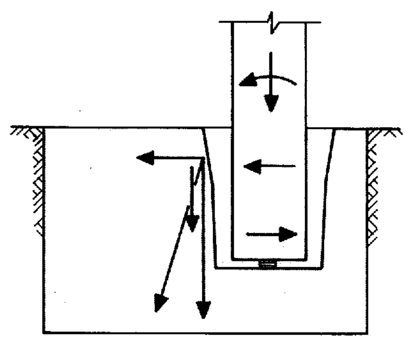


The design of the connections is the responsibility of the precaster. The foundation is the responsibility of the consulting engineer or main contractor, with the precaster supplying the design loadings and other information.

Columns in pockets In unbraced frames up to two storeys high, where base fixity is required for stability, it is common to detail the column so that it can be supported from a pocketed base. This type of connection is also used in many other frames.

This is the simplest detail for precasting, but it requires fairly large in-situ concrete foundations (Figure 6.20). The precast column may require additional links to resist the bursting pressures generated by end bearing forces. Vertical loads are transmitted to the foundation by skin friction and end bearing. The length of column embedded in the pocket may need a roughened or exposed aggregate finish, and the surface of the column within the pocket is sometimes provided with shear keys, depending on design requirements and the manufacturer's preference. The end may also have a reducing taper to increase restraint.

6:20 Typical pocketed base showing ultimate forces



The use of splayed ends, i.e. with the column widening towards the base, is mentioned in Reference 26. Splays can increase pull-out resistance, but the inclined surfaces reduce interface shear and complicate manufacture, so splayed ends are seldom used in multi-storey frames.

When there are overturning moments, the area of contact may be reduced by cracking at the joint between the column and the footing, but contact forces and interface shear increase, so the vertical load capacity is maintained.

Horizontal forces in the contact region are distributed within the pad footing by horizontal links, and particular attention should be paid to this if the distance between the pocket and the edge of the foundation is less than the smaller dimension of the column. The recommended minimum ratio for the depth of pocket to the width of column is 1.5 to 2.0, even though analysis may suggest lower values.

The concrete in the foundation around the pocket must be reinforced to cater for the forces transmitted from the column. This is the engineer's responsibility rather than the precaster's.

The specified strength of the grout used to fill the gap between the column and pocket is usually 40 N/mm^2

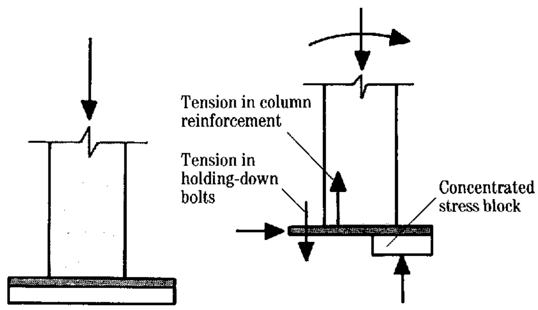
Columns on base plates Column base plate connections are used in many cases because they provide the column with immediate stability, and the depth of the foundation is not excessive. Figure 4.5, Section 4, shows examples of this type of connection.

The base plate details may be determined by production and/or structural considerations. A base plate of the same size as the column is often preferred because the plate can then be contained within the mould and the column length can be varied more easily during manufacture. The base plate can be larger than the column, however, and this may be necessary where a moment connection is required. Basic UK guidance is available in an ISE publication[26] for the design of base plates. This is similar to that given by the PCI.[20] Anchoring reinforcement is fitted through holes in the base plate and fillet welded on both sides. Where a moment connection is required, this mild steel reinforcement often controls the column design. Vertical loads are easily distributed through the grouted infill beneath the plate (Figure 6.21a).

The design of the holding-down bolts is affected by whether the connection is designed to be pin-jointed or to resist a moment. Bases subject to overturning moments require a greater attention to the detailing of both the precast column and the in-situ concrete foundation. Tensile forces are transmitted by bond in the precast column, bending and shear in the base plate, and tension in the cast-in holding-down bolts (Figure 6.21b). The foundation is designed for the resulting forces. Holding-down bolts are usually of grade 4:6 or 8:8, 20 mm to 30 mm in diameter and 375 mm to 450 mm long. The bearing area of the bolt head is increased by using a plate, nominally 100 mm square x 8 mm thick.

Because of the large compressive forces and bending moments that can occur, the maximum projection of the plate is usually restricted to about 100 mm, unless additional stiffening plates are used. The thickness of the base plate will depend on the load and moments to be carried, but typically it would be 15 mm to 35 mm. Pin-jointed footings can be designed by decreasing the in-plane lever arm. The desired effect can be obtained by using base plates with two bolts on one centre-line, or four bolts closely spaced.

6:21 Forces acting on base plates



a) Uniform stress-block with vertical load *b) With vertical load and moment*

Base plates of the same size as, or smaller than the column, are often used where the column is carrying mainly vertical loads, as in a braced frame, and where a projection around the foot of the column is structurally or architecturally unacceptable. This, however, reduces the cross-sectional area of the column, which may or may not be critical. With columns smaller than 400 mm square, these connections have a limited moment of resistance and are normally considered to be pin-jointed. To achieve an even better pinned connection, the group of holding-down bolts can be located in line with the main column reinforcement or positioned on the column centre-line. Short starter bars are welded to the base plate.

Columns on grouted sleeves The design of these joints (see Section 4, Figure 4.5) is identical to the grouted-sleeve splice described under *Column splices* (page 70) and should enable a compression or tension anchorage to develop between the precast column and the in-situ foundation. The diameter of the sheath must be correctly chosen to suit the size of the starter bars. Typically, a 40 mm tube is required for a 25 mm diameter deformed bar. The nominal cover to the tube and the minimum distance between tubes should both be at least 75 mm. Confinement links are provided in the column in the usual way.

When setting out the reinforcement projecting from the foundation, great care should be taken to ensure accurate alignment and to keep the bars vertical. The gap between the bars and the sheaths in the column must be completely filled by pressure grouting. Despite these difficulties, the joint can be made successfully and it may be considered to be monolithic. An additional benefit is that high-tensile reinforcement is used throughout.

Base joints for loadbearing walls or cladding
Wall-to-foundation joints serve essentially the same purpose as column-to-foundation joints except that, because of their large depth in the plane of stability, they transmit only vertical axial loads and horizontal shear forces. These connections are similar to those for columns, shown in Section 4, Figure 4.5. Ideally, the design details should be similar to the other joints used in the precast frame. For example, it is not generally expedient to use site welding for just one type of joint.

The large cross-sectional areas of cladding or wall units mean that stresses from gravity and imposed loads are usually small. The main difficulty is in achieving a detail that enables the in-situ foundation to be prepared without the need for great precision, but which produces a structurally sound connection. Grouted sleeve or welded plate-to-plate connections, as used for columns, are ideal for this.

Design of staircases
Precast concrete stairflights are designed as solid reinforced or prestressed concrete units. They may be manufactured tread-up, tread-down or on their sides (see Section 5). They may need to be designed for handling stresses and, if the soffit is cast face-up, they may need to be doubly reinforced. Normal compression reinforcement is often used to increase span/depth ratios. An economical solution for long-span units (exceeding 6 m) can be to use longitudinal precast reinforced concrete stringer beams, designed to support precast concrete transverse cantilevered steps.

Landings, whether simply supported or cantilevered, are usually designed as solid reinforced concrete slabs. Spans rarely exceed 4 m and it is often possible to design the units to fit within the depth of the floor units. Precast prestressed concrete flat planks or hollow-core units are sometimes used when an adequate connection can be made to the stairflight.

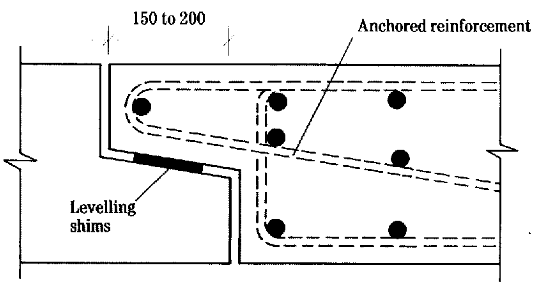
Staircase-to-landing connections Staircase-to-landing connections are often made using scarf or halving joints. These are designed as short cantilevers, where it is important to position the reinforcement very accurately. Small diameter bars at close spacing, say T8 at 100 mm centres, are preferable to larger bars, because they enable detailing requirements and cover to be achieved more easily. Figure 6.22a shows typical dimensions and reinforcement details.

Neoprene pads, or steel packing shims with a soft mortar bed, are used to ensure the correct bearing. Joints with structural continuity may be achieved by using intermittent scarf joints, as shown in Figure 6.22b. Welded connections made between fully anchored plates also may be specified. Concrete cover to the welding is provided for fire protection and appearance.

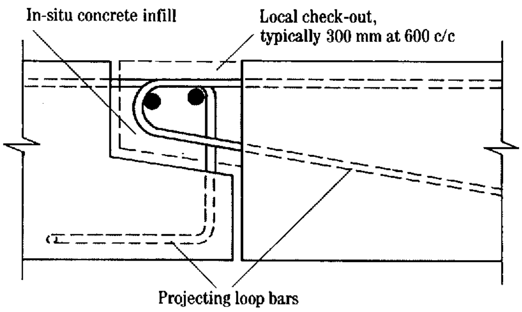
Shelf angles are an alternative to concrete scarf joints. Production can be simplified by providing cast-in sockets for the bolted connection. A small check-out is sometimes made in the supporting (landing) unit to receive the flange of the angle attached to the stairflight unit. At least two angles are used in the connection. The assembly is designed to carry the vertical load from the stairflight acting at the most onerous position. Flexural and shear stresses at the root of the angle are considered, and pull-out forces and shear stresses are used in designing its anchorage.

The friction grip bolts used to make the connection require slotted holes to allow for fixing deviations. Alternatively, as shown in Figure 6.22c, an angle with welded-on anchorage reinforcement may be cast directly into the stair unit. The steel angle needs a concrete cover or other method of fire protection.

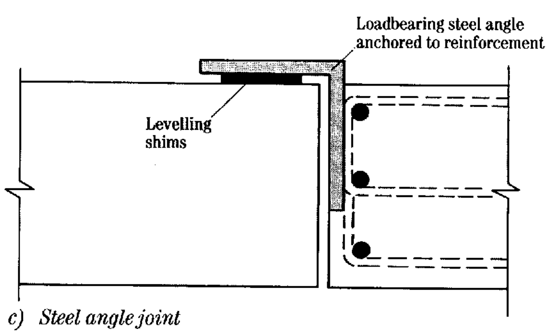
■ **6:22** **Staircase-to-landing connections**



a) Continuous scarf joint



b) Intermittent scarf joint



c) Steel angle joint

DESIGNING FOR HORIZONTAL LOADS

Introduction

Horizontal forces in precast concrete frames arise mainly from wind loading and temperature gradients. These horizontal forces must be resisted and transmitted through the components and their connections.

There is an increasing awareness of the need to ensure the structural integrity of joints in pre-fabricated construction. It is not always realized that the details used to achieve robustness in a frame have a significant effect on the structural mechanisms by which horizontal forces are distributed.

Two key factors need to be satisfied in design: horizontal load transfer by diaphragm action in the precast concrete floor plates, and stability in the vertical bracing elements. In the latter, there is an obvious difference between unbraced and braced frames.

Horizontal load transfer

Diaphragm action in precast floors Horizontal loads from wind or earthquakes are usually transmitted to moment-resisting frames or to shear walls, by the roof and floors acting as horizontal diaphragms (Figure 6.23). The precast concrete floor or roof diaphragm is analysed by considering the slab to be a deep horizontal beam. This is analogous to a plate girder or I-beam containing chord elements, as shown in Figure 6.24a. The structural walls or frame act as supports for this analogous beam with the lateral loads being transmitted to them.

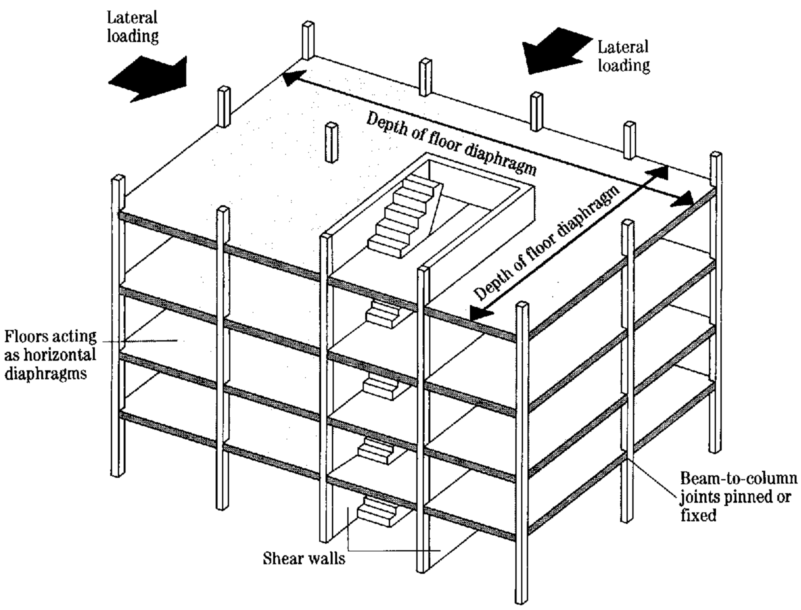
Floor systems, consisting of individual precast concrete units, are used as the diaphragms. Hollow-core units (see Section 4, Figure 4.12) are the most widely used in this context, because of their edge profile and the in-situ concrete in the longitudinal joints between adjacent units. These joints are normally uncastellated and unreinforced and, in spite of a slight roughening of the edges which can occur during manufacture, they may be considered as plain.

The most critical situation is where the floors span parallel with the shear walls. The shear is transferred between adjacent floor units by compressive stresses, aggregate interlock and dowel action. These forces are inseparable because the reinforcement in the in-situ strips at the ends of the units provides the clamping forces necessary to generate the aggregate interlock (Figure 6.24). An ultimate interface shear stress of 0.23 N/mm^2, as given in BS 8110,[8] is commonly used in design. Recent test results[38,39] have shown that the interface shear stress generated at the edge joint between hollow-core units is in excess of the working load by a factor of at least 2.15, even when there is an initial crack up to 0.55 mm wide between the units.

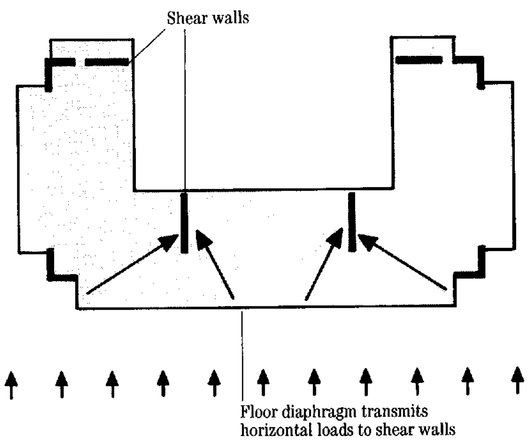
The shear stress, calculated on the effective depth of the joint (typically taken as D − 30 mm, where D is the depth of the slab), is seldom critical, so that hollow-core units normally provide sufficient diaphragm action without a structural topping, but the units must be restrained from moving apart. The transverse tie reinforcement needed to achieve the necessary ultimate interface shear stress can be readily determined from BS 8110.

Structural toppings are not normally necessary to achieve diaphragm action in the floor. Generally they are only necessary where composite action with the floor units is required (Figure 6.4b), or where there are heavy concentrated loads such as those from storage racking and heavy machinery, or moving loads such as those from forklift trucks. A structural topping will also be necessary to provide diaphragm action when the floor would otherwise be unable to transmit in-plane shear. Structural

6:23 Diaphragm action of floors within a structure



a) Floor units form diaphragm

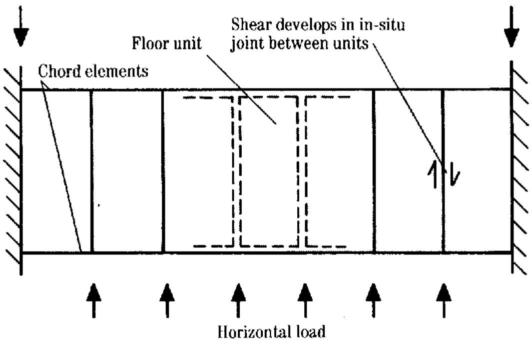


b) Loads transmitted to shear walls

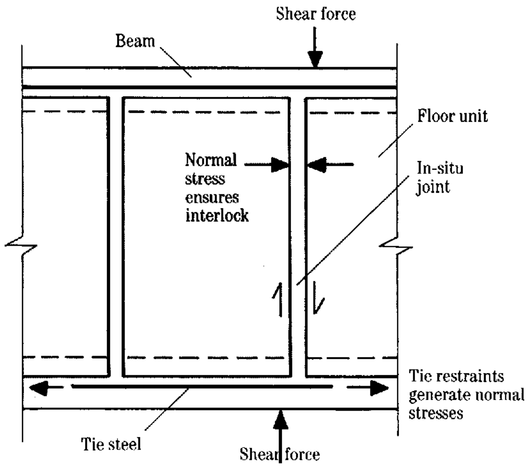
toppings should always be reinforced with a light fabric.

The controlling factor in buildings exceeding eight or nine storeys, where the full ultimate stress is required, is likely to be the amount of tie reinforcement that can be accommodated in the in-situ strips between the end of the floor units and the supporting edge beams.

6:24 Diaphragm action in precast floors



a) Analogous plate girder



b) Force mechanism

Diaphragm action in composite floors Generally, composite floor systems are designed on the basis that the precast concrete floor units provide restraint against buckling in the relatively thin in-situ reinforced concrete topping, with the shear being carried entirely by the topping. The shear strength of the welded connections between double-tee units may be included, but it is generally ignored. The design ultimate shear stress is normally taken as at least 0.45 N/mm^2 (25 grade concrete) and the effective depth of the topping is measured at the crown of prestressed flooring units, where the topping is thinnest. To produce continuity, reinforcement in structural toppings is extended into the shear walls. The shear capacity of the in-situ topping is unlikely to be the governing factor in the frame layout. Care must be exercised in the design of such floors, particularly where there are openings adjacent to external shear walls, or to other elements which provide stability.

Design of frames for horizontal loading

Unbraced frames The stability of unbraced pin-jointed frames is provided entirely by columns designed as cantilevers for the full height of the frame (Figure 6.1b). It is normal to ignore partial restraints provided by moment-rotation or torsional stiffness in the beam-to-column connections, deep external spandrel panels, or internal brick or block walls. The line of load application is through the centroid of the flooring system. The distribution of horizontal loading between columns is directly proportional to the second moments of area of the columns in the cracked condition (i.e. using the modular-ratio method) at the serviceability limit state.

The torsional stability of non-symmetrical buildings is considered, but the positions of columns are usually such that the shear centre of the system coincides fairly closely with the centre of pressure. The frames, as with other forms of construction, are designed for wind loading in accordance with CP3, Chapter 5, Part 2[40] and force coefficients are determined for the shape of the building. Where appropriate, an ultimate horizontal loading of 1.5% of the total dead load of the structure replaces the ultimate wind force, in accordance with BS 8110. Ideally, the design loadings, including wind, should be given in the tender specifications (see the Appendix). The maximum height for an unbraced pin-jointed frame is about ten metres. The factors controlling heights greater than this are likely to be architectural restrictions on the sizes of columns, and/or the magnitude of the moment-restraint required at the foundation. The moment carried by the column depends on the degree of fixity between the column and the footing, and on the resistance of the soil to rotation of the footing.

Where the foundation is specifically designed to carry moments, BS 8110: Part 2 allows the effective length of the column to be determined by taking the total stiffness of the footing and soil as equal to that of the column ($\alpha_c = 1.0$). The PCI method[20] is more explicit, but the result is approximately the same.

Braced frames Braced frames are the best means of providing stability in multi-storey construction, irrespective of the number of storeys (Figure 6.1a): connection details and the design and construction of foundations are greatly simplified. Precast concrete shear walls are inexpensive, have large in-plane stiffness and strength, are easy to erect, and may be integrated with the frame as either infill walls or cantilever walls or boxes. These have been described in Section 4. Infill masonry walls and steel cross-bracing are also used.

Diaphragm action and a centroidal line of load application in the floor plate is once again assumed. The distribution of horizontal loading between shear walls depends on a number of factors.

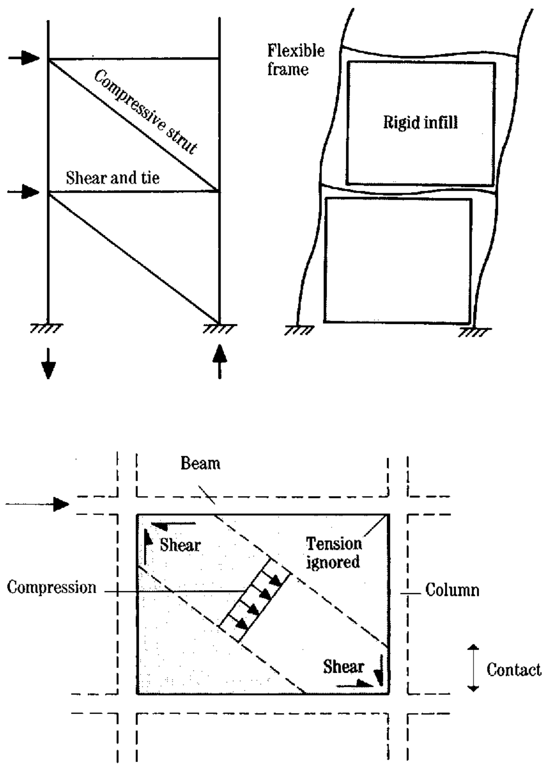
- In-plane deflection response – this is predominantly a flexural deflection in cantilever walls, a shear deflection in infill walls and a truss deflection in steel cross-bracing.
- Position – ideally, the frame should be balanced by positioning the walls according to their stiffnesses and in such a manner that the centre of pressure of horizontal loading lies approximately at the shear centre. Torsional effects, however, can be designed for.
- Connections between walls, or between walls and foundations – walls may be used in the structure for purposes other than for stability, such as to support a half-landing, and so designed as not to be moment- or shear-resisting. This may be achieved by under-reinforcing the connections, which allows loads to be redistributed away from the wall whilst maintaining structural integrity.
- Expansion joints in the floor diaphragm – expansion joints are usually provided at about 80 m intervals in the floor diaphragms if the frame is rectangular on plan, or at about 60 m intervals if the plan is non-rectangular. The distance between such joints will, however, be influenced by the stiffness of the beam-to-column joints, and larger intervals may be possible.

Design of infill shear walls Most of the pioneering work on infill frames – albeit using masonry infill – was carried out by Stafford-Smith and Carter,[28] Mainstone[41] and Wood.[42] The design procedures suggested by these authors are widely respected and used in the UK. Infill shear walls rely on composite action with the pin-jointed column-and-beam frame for their strength and stiffness. Where an infill shear wall is built solidly into such a frame, its composite action with the frame considerably increases its resistance to horizontal loading (Figure 6.25). This is similar to the behaviour of stiff beams on elastic foundations, in that the resistance to horizontal loading varies with deformation. One of the most important factors is the quality of the shear key connection which, for manufacturing purposes, is usually unreinforced.

There are usually a number of design assumptions for infill shear walls. These are listed below.

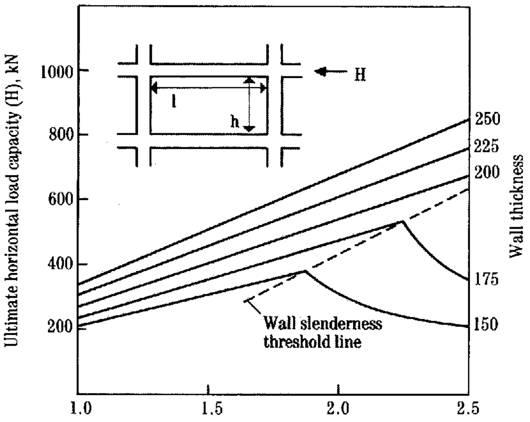
- Ultimate horizontal forces are resisted by a diagonal compressive strut across the concrete infill wall, and tension and compression in the columns. The effective width of the strut depends primarily on the relative stiffnesses of the wall panel and frame, and on the geometry of the panel. This is not critical in concrete walls because of their strength in compression.
- The diagonal tensile strength of the reinforced concrete wall is ignored, but the amount of reinforcement is sufficient to prevent excessive diagonal cracking and to maintain the intrinsic shape of the wall panel, particularly the shape at the corners.
- Slender wall panels are designed, to BS 8110, as slender braced plain concrete walls (the reinforcement ratio is usually less than 0.4%).
- The shear resistance at the horizontal interface between the beam and the wall panel is also based on shear in plain concrete walls, typically using an ultimate average stress of 0.45 N/mm^2 for dense aggregate concrete, grade C25, or a value equal to 25% of the vertical precompressive stress.
- The wall is not subjected to vertical frame loading, i.e. floor beams are assumed to be structurally isolated from the wall units, even though the gap between them is grouted solidly.

6:25 Behaviour of frame with infill shear wall



Load transfer occurs between the wall and frame. The length of contact gives the effective width of the compressive strut and, together with the appropriate reductions for slenderness and eccentricity, the ultimate diagonal compressive strength of the wall is computed. If the permissible horizontal shear stress is not exceeded, load v. infill shear wall size may be presented as shown in Figure 6.26. Vertical column reactions, both compression and tension, are taken into account in the column design, as appropriate. Except for self weight, no vertical reactions from the wall are carried by the beam.

■ **6:26 Horizontal load capacity of precast concrete infill shear walls**



Values based on h = 3.0 m and f_{cu} = 40 N/mm²

Design of cantilever shear walls and shear boxes Cantilever walls (Figure 6.27) and boxes can be designed to work with precast concrete frames without any problem. This confidence is based, particularly in the case of shear boxes or interconnected shear walls, on the size of these units and on the extensive use of continuity reinforcement or other mechanical connections between the units. The elements are designed as reinforced concrete walls in the conventional manner.

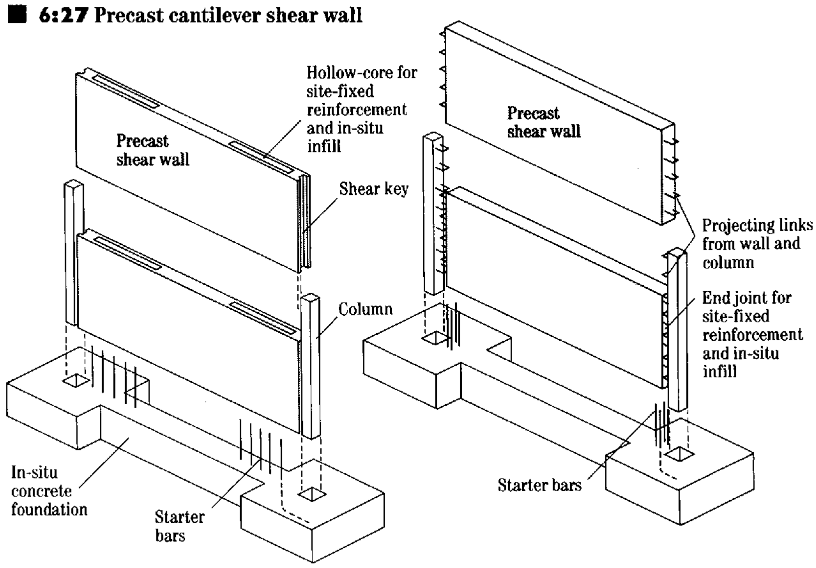
The slenderness of the wall, out-of-plane loading and constructional tolerances are all taken into consideration. Particular attention is paid to the reinforcement at corners – especially to the reinforcement needed to avoid cracking when lifting the units at an early age – and around windows and doors. Large openings, such as those for doors and services, are readily included by considering alternative load paths for the vertical and horizontal forces.

Horizontal connections are made at floor levels. Fully anchored vertical reinforcement, or other mechanical devices such as welded or bolted plates, is used to transfer the tensile forces between the wall units, and the precompression across the faces of the joint between the units is maintained. The joint is assumed to be cracked only in flexure and the shear friction theory is used to justify taking the full length of the wall into consideration when determining the average ultimate shear stress. The normal limiting value is 0.45 N/mm^2 for 25 grade concrete.

Units to be assembled from individual flat panels are connected on site using hollow-core walls (Figure 6.27a) or in-situ vertical end joints (Figure 6.27b). The resistance to sliding depends on the size and shape of the joint and the amount of reinforcement projecting from the adjoining panels. Alternatively, mechanical connections may be used to produce immediate stability in the walls: fully-anchored plates provide the site welder with easily accessible down-hand welding. In some cases, vertical shear keys between panels, or between panels and columns, may be sufficient.

A deep beam analogy is used in design because the wall behaves as a cantilever. The precast elements are more expensive to manufacture than plain solid walls, but this solution may find favour in seismic regions where ductility is important.

■ **6:27 Precast cantilever shear wall**



a) Hollow-core walls *b) Vertical end joints*

Partially-braced frames Partially braced frames are used where stability walls are architecturally undesirable in the upper two (or a maximum of three) storeys. The principles were shown earlier in Figure 6.1c. The frame is designed as fully-braced up to a specified level. This need not be the same level throughout the building as stability requirements may vary in different directions (see Section 7). The columns are cantilevered above this level, as in an unbraced frame. The advantages in using this method are listed below.

- There are no bending moments at the foundations.
- Columns are braced between ground and first floor, where greater headroom is often required.

In most one or two storey frames, column sizes and reinforcement are no greater than if the frame were fully braced, i.e. stability is being obtained at no extra cost.

- Column splices may be made in the braced part of the frame.
- In the unbraced regions in the upper floors, clear floor areas for open plan offices and staircases are punctuated only by columns; clear areas of up to 15 m x 9 m are feasible using double-tee roof slabs.
- The shear wall area is minimized.

Columns in the unbraced part of the structure are designed as cantilevers with an effective length factor of between 1.6 and about 2.0, depending on the amount of fixity present. Shear forces at the bases of the unbraced columns are carried by the floor diaphragm and distributed to the stiffening components in relation to their stiffnesses and positions. However, bending moments from sway in the unbraced parts of the frame are carried over into the braced parts. The effective length factor for the braced columns in this region is 1.0.

Summary of frame types Table 6.1 shows the typical economical range of frame types and associated bracing elements.

Table 6.1 Economical range of frame types

Type of frame	Approximate number of storeys	Bracing elements
Unbraced	2 up to 3	Columns Columns (roof load small)
Braced	up to 4 up to 5 3 – 6 3 – 10 10 – 15 15 – 20	Steel cross-bracing Brick or block infill walls Precast concrete wind posts (deep columns) Precast concrete cantilever shear walls Precast concrete shear boxes In-situ concrete shear core
Partially-braced*	4 – 5 5 – 10 5 – 12	Steel cross-bracing Precast concrete infill walls Precast concrete cantilever shear walls

* Top 1 or 2 storeys unbraced.

STRUCTURAL INTEGRITY AND DESIGN FOR ACCIDENTAL LOADING

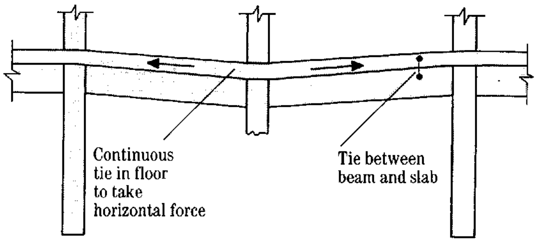
Resistance to progressive collapse

Because there is some lack of continuity in the joints of precast concrete frames, it is sometimes thought that, under certain loading conditions, they do not have the ductility inherent in monolithic construction. There is little evidence to support this view, as an inspection of joint details, specifications and modern construction practice would show. In fact, precast concrete frames have considerable robustness resulting from careful design and detailing, and experience in site fixing. Based on extensive experience of their use and performance, precast concrete frames can be designed for earthquakes and other abnormal conditions. This is covered in a PCI technical report.[43]

The integrity of precast concrete construction is also supported by tests carried out at the Building Research Establishment on precast concrete floors and beams.[44] It was found that if the continuity reinforcement over a missing support was correctly positioned, deformations up to 20% of the adjacent single span could occur without collapse.

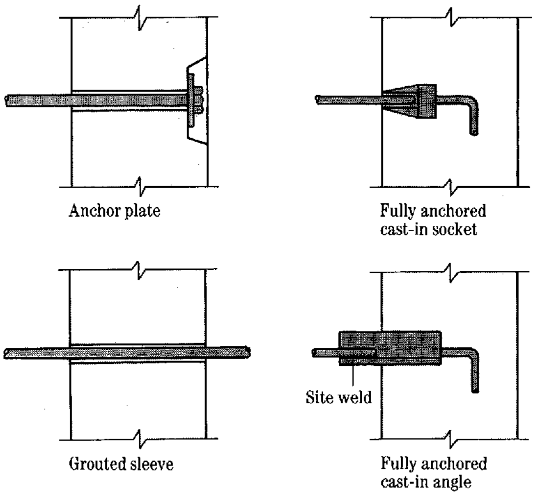
Horizontal ties Structural integrity, in the context of avoiding progressive collapse, is normally obtained by using the fully-tied solution (Figure 6.28), rather

6:28 Catenary action to prevent progressive collapse



than the alternative load path or protected key elements solutions. The requirements for horizontal ties in BS 8110: Part 1 are satisfied by using continuous ties in the in-situ concrete strips, or by using ties partly in the in-situ concrete and partly in the precast concrete components. In either case, continuity between the ties and the precast components is obtained by lapping or welding reinforcement, or by using threaded couplers, cast-in sockets or other anchored fixings. Some of these methods are shown in Figure 6.29.

6:29 Anchorage and continuity ties



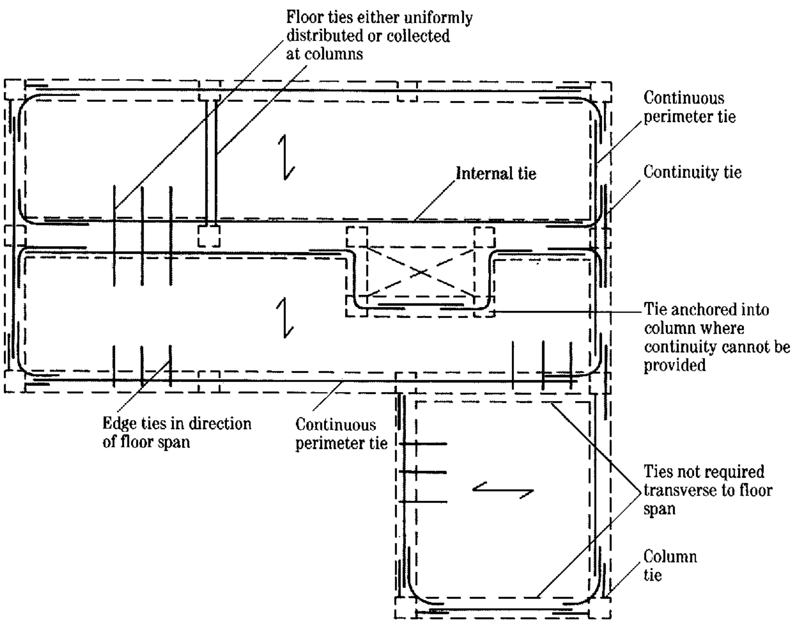
Ties passing through precast concrete columns or walls are fed through oversized sleeves, usually 2½ times the diameter of the tie bar(s), and later grouted in. Peripheral ties are made continuous round external corners by concreting them into the in-situ edge joints (45° splayed corners may be necessary), or by using threaded couplers or anchored plates. It is difficult to place a single tie bar around an internal corner, so continuity is normally obtained by anchoring the bar to fixings in the columns. Continuity is not usually provided in precast concrete staircases, because these units are not normally essential to the stability of the frame, but Figure 6.22b shows how continuity between precast concrete staircases and their supports might be achieved if it is required. Non-structural cladding panels are not tied into the structure for the purpose of providing frame stability.

Internal ties parallel to the span of the floor are either distributed evenly using short lengths of tie steel anchored by bond into the opened cores of the hollow-core floor units (Figures 6.6 and 6.7), or grouped in full-depth in-situ strips along the column grid line. In the first case, the mechanism relies on the generation of an adequate pull-out force in the hollow-cores filled with in-situ concrete. Tests have shown that this has a factor of safety in excess of 2.2,[33] based on a maximum tie force of 60 kN/m. With composite floors, the ties may either be cast into the units or into the in-situ topping. Internal and peripheral ties parallel to the span of the beams are grouped at the beams. To prevent them from splitting out from the in-situ strip, these ties usually pass beneath the floor ties or are placed inside projecting loops, hooks or similar projecting beam reinforcement.

The in-situ strips over the tops of beams have a minimum depth of about 50 mm. Because 10 mm size aggregate is used, this exceeds the BS 8110 requirement for tie bars up to 20 mm diameter. The tie bars may be mild steel, high-tensile deformed bar, or prestressing strand. Strand is an attractive alternative to high-tensile steel bar because of its favourable mechanical properties and the long lengths available on site.

The general principles of providing continuity are shown in Figure 6.30, which also shows the distribution of ties in a typical frame layout without a structural topping.

6:30 Horizontal stability ties



Vertical ties Continuous vertical ties are provided in all precast frames, regardless of height (including frames of less than five storeys). Column splices (Figure 6.18), and horizontal joints between loadbearing wall panels are designed to ensure that an adequate vertical tie force is generated.

7

SITE ERECTION

EFFECT OF CONSTRUCTION ON DESIGN

General considerations

To ensure that all those involved with the construction of the frame are aware of the method of construction, the design office should pass to the site detailed instructions for the sequence of erection. With a consultant-designed frame, this responsibility will rest with the design engineer. Details of any variations required on site must be passed back to the design office.

This is a long-established practice with the major frame producers. Many years' experience have created the understanding between the manufacturers' engineers and site fixing gangs which is one of the major factors contributing to the success of precast concrete frames. Attention should be paid to safe practice, and details of this are given in Reference 45.

All accredited precast frame producers supply, on request, written statements of the principles of site erection, methods of making structural joints and specifications for materials. These are generally in accordance with the requirements of BS 8110: Part 1, with regard to:

- Critical jointing details, materials and methods.
- Critical dimensions, allowing for manufacturing and site tolerances.
- Temporary propping or fixings.
- Advancing the construction ahead of completed and maturing sections of the building.

The design for temporary conditions should take into account overall frame stability and the stresses in individual frame components and joints. Checks should be made to ensure that the units can be handled, both in the factory and on site, without causing any premature failure or significant defects. This is particularly important in units, such as stocky plain walls or lightly loaded columns, where only small amounts of reinforcement are required to satisfy serviceability requirements. In these cases extra reinforcement is provided to cater for lifting and erection.

Load paths through a partially completed structure may be different from those in the completed frame. An example is the temporary state when floor units have been placed on one side only of an internal beam. Here the connection should be checked for its resistance to torsion and, if it is found to be inadequate, props should be placed under the beam to prevent rotations from damaging the connection. This is discussed in more detail later.

Designing for lifting and erection

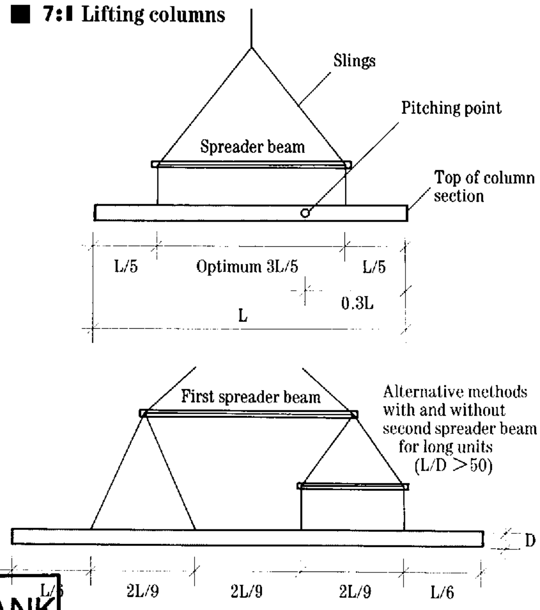
In a precasting works, concrete strengths tend to be governed more by the need to lift the units from the moulds than by the specified ultimate strength. As little as three days may elapse between stripping the moulds and transporting the components to site. Some components, such as staircase units which might otherwise have only a single bottom layer of reinforcement, may need to be doubly-reinforced to control stresses in the 'green' concrete. Bending moments and shear forces caused by lifting and erection are calculated. Typically, the self-weight is increased by 25% to allow for mould suction and impact, as well as for any site construction loads.

Lifting points are selected so that the flexural and shear reinforcement provided for in-service conditions is fully utilized when lifting. The lifting points are also chosen to minimize deflections. It is not possible to eliminate flexural effects, but any slight cracking in the components is usually neither structurally nor architecturally significant, nor detrimental to the in-service behaviour.

The positions of the lifting points will depend on the type of component, and on the distribution of its reinforcement. Typical positions for symmetrically reinforced members are shown in Figure 7.1. Positions for units where the reinforcement is principally located in one face are shown in Figure 7.2. Flat wall units and longer beam components may require four-point lifting. Additional reinforcement is required around the lifting points.

Most precast components are laid horizontally for delivery to site, although some special cladding or wall panels are delivered near to the vertical on A-frames. Columns are usually raised to the vertical on site using a pitching point at 0.3 times the height of the column from the top (Figure 7.1), but multiple points may be required for some units.

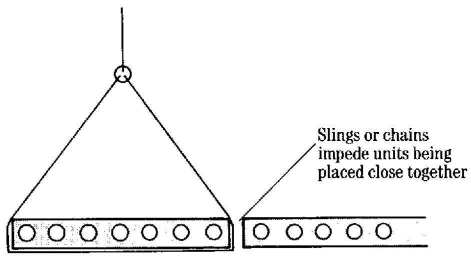
7:1 Lifting columns



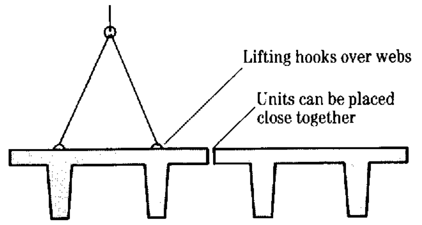
Hollow-core floor units are a special case because lifting hooks are rarely built-in during manufacture, and the tension caused by hogging moments from the prestress must be considered when the lifting points are not close to the ends. The units are usually lifted on site by using slings or chains (Figure 7.2a), or specially designed clamps. The design of these units is rarely governed by handling stresses, except for very long spans. Double-tee units are hoisted using four lifting points, each over the lines of the webs (Figure 7.2b). Details of the reinforcement around these points may be obtained from the manufacturer.

Because of their shape, wall units and cladding panels are pitched from the top, which may be the most onerous design condition. Lifting points may be located vertically over the centre of gravity of the unit (Figure 7.3a), or offset to allow the unit to tilt slightly inwards at the top or bottom to assist positioning (Figure 7.3b). Special lifting devices

7:2 Lifting floor units

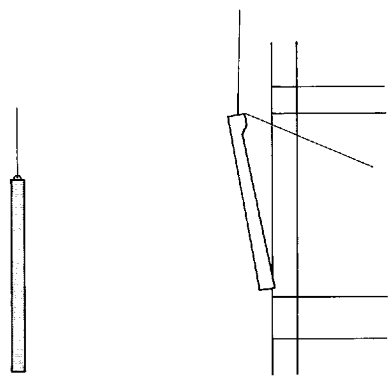


a) Hollow-core units



b) Double-tee units

7:3 Lifting wall panels



a) Hooks positioned for axial lift

b) Hooks offset to assist positioning

have been evolved to ensure that the concrete is not damaged during erection.

ERECTION PROCEDURE

Figure 7.4 shows the stages of erection. In this example columns placed in pockets are lowered onto a pre-levelled plate giving a clearance of between 50 mm and 75 mm to the sides and bottom of the pocket. Temporary wedges are driven into the gaps and, together with diagonal props, are used to line and plumb the column. In-situ concrete, with an early strength of about 40 N/mm^2 at five days, is compacted into the gap to ensure that a structural connection is made before the first floor units are placed. The props remain in position until the first floor is complete and each column has been tied into the framework.

Columns founded on base plates can attain a more immediate fixity, but it is not considered that a structural connection has been made until the in-situ grout underneath the plate has reached sufficient strength.

The next components to be fixed are the other vertical loadbearing elements, such as shear walls, which rely on the columns only to fix their position and not for any structural purpose. These are placed on levelling shims which are left in place. Where construction joints are narrow, i.e. less than about 12 mm wide, they are unreinforced and dry-packed with a cement-sand mortar. Wider joints are filled with concrete containing a small aggregate, typically 6 mm or 10 mm. Columns have temporary fixings, or diagonal propping is used until the in-situ connections have matured.

Fixing procedures for beams depend largely on whether the connections:
- Provide an immediate restraint to shear and torsional forces, e.g. bolted cleats, or billets with a bolted or welded top fixing.
- Do not provide this restraint, e.g. dowelled corbels and billets with a top fixing of an in-situ tie.

For the latter, temporary propping is used because the beam is unstable until the in-situ grouted connection has matured. A grout strength in excess of 20 N/mm^2 at three days ensures stability at an early stage. In all cases, the shear capacity of the connection is not affected by the erection sequence and is not dependent on the strength of the infill.

Spandrel beams, deep beams, i.e. those exceeding 900 mm deep, and beams with a non-symmetrical cross-section, are provided with a second fixing to improve temporary stability. The fixing may be protected by the grout, or may remain exposed, in which case it is made from stainless steel or other corrosion-resistant material.

Precast concrete components with a large plan area, such as floor units and staircases, present few fixing problems provided that there is an adequate bearing and the unit can be lifted near to the horizontal. With units on continuous bearings, such as is the case with hollow-core floor units, uneven or cambered beams or other supporting components may cause rocking where the units are more than about two metres wide. This problem is not widespread in the UK, because most units are narrower than this, but where it does occur, it is overcome by bedding the units on mortar.

Floor units are not propped unless the structure is designed as a composite structure utilizing propping forces. Hollow-core units are lifted using chains or slings. This means that they cannot be placed directly side by side, so they are usually levered into position using the edge of an adjacent unit as the fulcrum. Small aggregate concrete infill, with a high workability, is compacted around the perimeter of each unit. This is done in a single pour over the largest possible floor area compatible with stability.

Double-tee units are usually larger and their fixing requires careful attention. Four-point lifting enables the units to be accurately located. When a single bay of flooring is complete, cumulative tolerances are checked, the units are adjusted if necessary, and site welding is started.

On-site connections

In the design of connections to meet erection requirements, the most important factors are:

- Accessibility.
- Temporary stability.
- Tolerances.

The correct erection procedures must be followed to ensure that cumulative dimensional deviations from manufacture and site erection do not cause the connection to fail during construction. The connections must permit components to be lowered unhindered onto their supports and there must then be adequate access to the connection. A slewed approach is sometimes permissible in difficult situations, but is generally best avoided.

Connections may be:

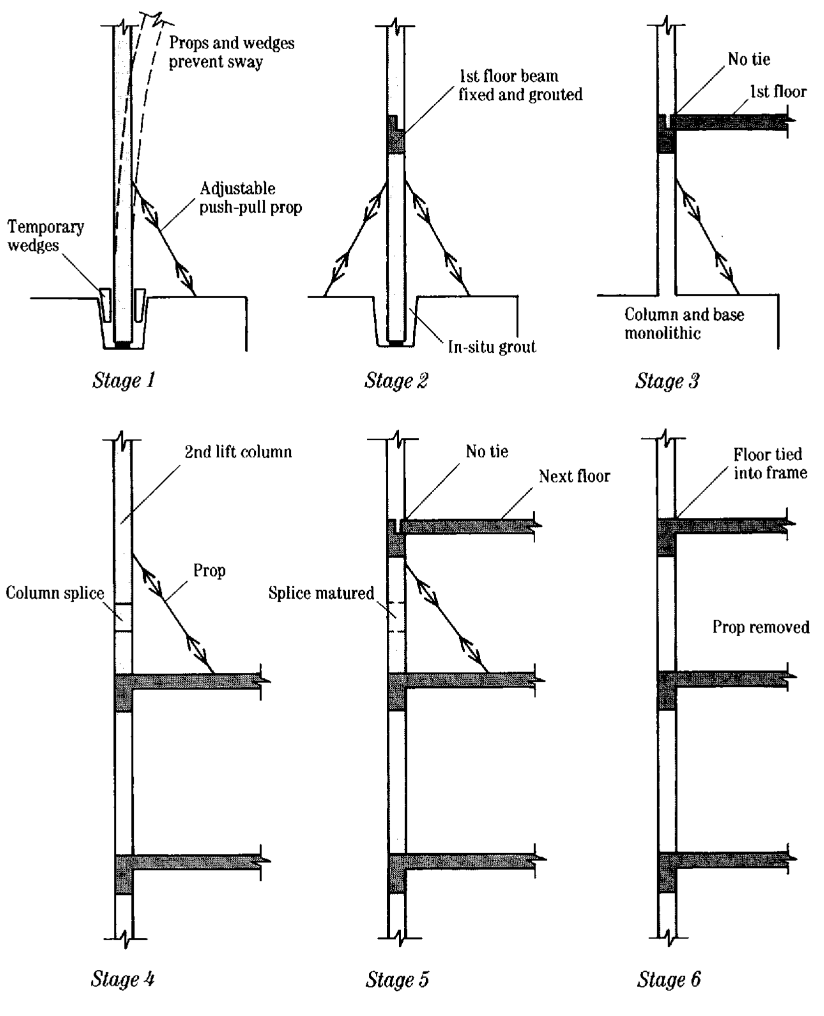
- Designed for the permanent condition, e.g. beam-to-column connections.
- Designed to assist erection, e.g. temporary wall cleats.

The latter are the sole responsibility of the precast contractor and, on the assumption that failure of these connections would not impair the stability of the completed frame, they are not covered in this publication.

Structural joints between precast concrete components require only small quantities of materials. Small aggregate concrete, cementitious mortar, epoxy mortar and adhesives must be carefully specified and accurately placed and compacted. The ISE design manual on joints[26] is often used by designers to assess proposed jointing details.

In the temporary condition, beams are stabilized, either by propping, or by using a permanent welded or bolted connection in the ungrouted state. One onerous situation is the asymmetrically loaded spine beam shown in Figure 7.5. This produces a torsional moment which must be resisted by the connection or prevented by propping. With the exception of welded plate connections, where propping is the preferred method of stabilizing the beams, most connections in a typical frame are capable of generating sufficient restraint. Similar problems occur in some of the wider L-shaped edge beams and in special cantilevered sections, but in general, these are easily overcome by propping, or by an extra top fixing cleat.

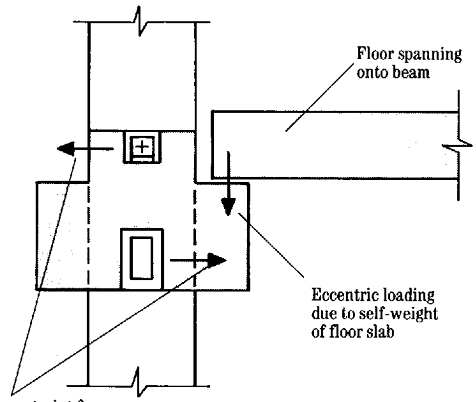
7:4 Precast frame erection sequence



Beam-to-column connections are designed with sufficient tolerance to enable the beam to be levelled accurately using shims. It is important for the shims to be located accurately to prevent large eccentricities at the column face. The shim should also be large enough to prevent local bearing problems, because it cannot be assumed that any in-situ grout placed around the connection will penetrate to the bearing surfaces. Similar principles should be applied when using neoprene pads between concrete surfaces, such as on column corbels. Where the packing is very thick, a sandwich construction of a steel plate between two neoprene pads is preferred.

Shear walls are temporarily stabilized by diagonal propping, or by using temporary or non-structural permanent cleats or plates which are later grouted into the structural joint. These non-structural permanent fixings are becoming increasingly popular. Out-of-plane forces generated by these fixings are usually very small and their effect on design is generally not significant, but this may not be the case where loadbearing cladding is used extensively. Here, the temporary condition is critical, especially on exposed and windy sites, and the panels must be stabilized without detriment to the remainder of the structure, which may not have reached full maturity. A range of bolted and welded connections is used in these situations.

■ **7:5 Asymmetric loading on spine beam**



Strength and maturity of connections

Bolting or welding provides the connection with some immediate strength, although in-situ concrete may be required for the joint to achieve full service strength. If this is so, the strength of the partially completed joint is checked against the temporary loads and, if necessary, the construction sequence is interrupted and work carried out elsewhere until the in-situ concrete is sufficiently strong. Temporary propping is not always required during this time. In-situ concrete and grout are designed to have early strengths and low shrinkage. Typically, a strength of about 10 N/mm^2 is specified to cater for temporary loading. This strength is usually reached within two days.

Other important connections are:

- Column splices, where the joint must achieve an adequate compressive strength before upper storey beams and slabs are erected.
- Floor slab joints, where in-situ concrete infill is used to develop the floor diaphragm which transfers horizontal loads to stabilizing elements.
- Shear wall joints, which must be made in correct sequence behind erection.

DESIGN FOR TEMPORARY FRAME STABILITY

The design of frames for temporary conditions depends largely on the site erection sequence, so it difficult to give general recommendations. The sequence of erection is controlled by many factors. The most significant of these are the structural form – such as the plan shape and whether the frame is unbraced or braced – crane accessibility, and the positions of stability walls.

Unbraced frames are inherently stable as soon as the moment connections at the foundations have been completed, but this does not mean that temporary stability may be ignored. A fully-tied first floor slab must be in place before temporary props are removed from the columns, and the verticality of the columns should be checked as each floor level is completed. Side-sway can occur only by the simultaneous lateral movement of all the columns tied into the frame. Because absolute rigidity cannot be achieved, the function of bracing is to limit side-sway to within acceptable limits, which are normally well below site fixing tolerances.

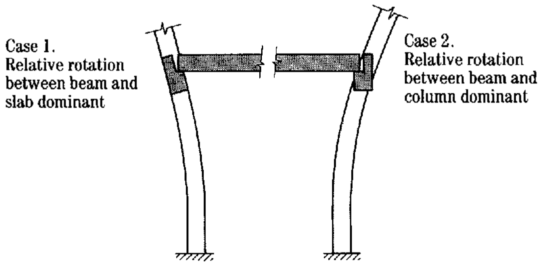
In braced frames, which rely on stabilizing elements, construction starts in an area that contains at least one stabilizing element in each orthogonal direction and work then proceeds outwards and upwards from there. If this is not possible, stability is provided by temporary steel cross-bracing or diagonal propping. There is a degree of redundancy because most pinned bases have a partial moment stiffness which may be called upon to assist stability, in the temporary condition, up to the level of the first column-to-column splice. Exceptions to this are columns founded on retaining walls or on in-situ ground beams.

Wind loading and eccentric loading caused by overhanging components or lack of plumb, are considered at this stage. The forces on open frame-works are generally not critical unless the frame is partially clad at a level where the floor plate is incomplete. In this case the line of columns on the windward and leeward external faces are checked against forces calculated using local wind pressure coefficients and concrete strengths appropriate for the age of the components.

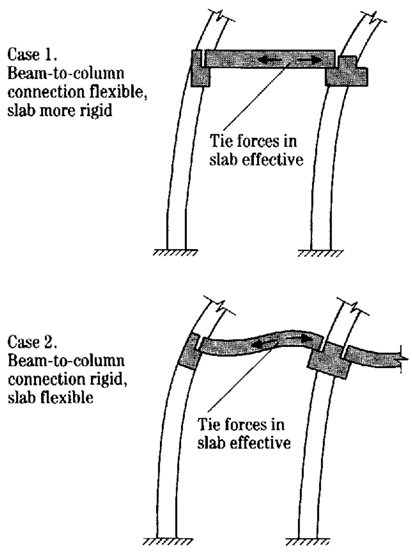
Eccentric loading from non-symmetry is more easily calculated and allowed for. The main modes of possible instability are summarized in Figure 7.6. The divergent mode (Figure 7.6a), is possible where particularly large concrete cladding panels are fixed out of sequence, or are fixed too early in relation to the maturity of the structural frame. This may occur in cold weather where in-situ concreting is not permitted and the contractor is anxious to continue with erecting the precast units. These situations should be anticipated by using temporary tie-backs.

In the sway mode (Figure 7.6b), rotations between a beam and slab, and between a column and beam, are both possible in the temporary condition. Props are used to prevent side-sway until the joints are grouted. The inertia of most types of slabs with depths greater than 150 mm, as used in multi-storey frames, is sufficient to prevent the type of side-sway shown as Case 2 in Figure 7.6b.

■ **7:6 Potential instability modes during erection**



a) Divergent mode



b) Sway mode

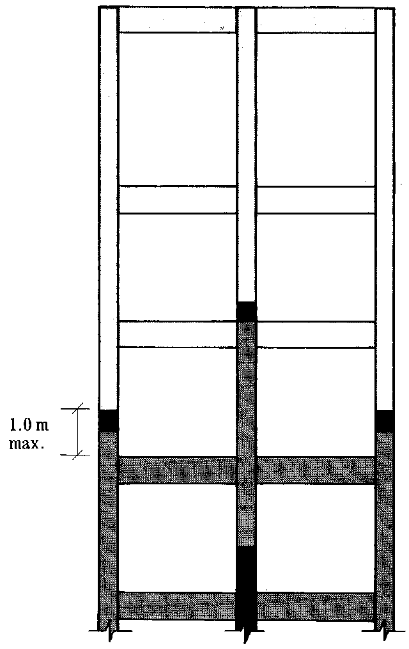
Lack of plumb is largely eliminated by close fixing tolerances, although frame movements caused by temperature effects, shrinkage and self-weight will inevitably cause dimensional variations of about ± 10 mm in 60 m. Although it is not generally realized, precast concrete columns are flexible and easy to straighten. One of the benefits of precast concrete frames is that they are vertically aligned at each floor level before the in-situ concrete ties between the beams and floor are cast. Push-pull props are used extensively for this alignment (Figure 7.7), and fixings for these, such as threaded sockets and brackets, are supplied fully anchored into the precast units.

By staggering column-to-column splices in alternate rows of columns (Figure 7.8), construction proceeds with at least half the number of columns cantilevering above a critical level. The strength of these columns considerably assists erection by providing stiffened points which can be used to brace

■ **7:7 Propping of frame during erection**

subsequent lifts of the frame. The axial force and bending moment resistance of some of the smallest column splices, such as for a 300 mm x 300 mm column, is sufficient to permit a sway of about 15 mm at the top of a two-storey column. This is unlikely to be exceeded.

■ **7:8 Staggering of column splices**



The erection of the frame should not get more than two storeys above the level at which the frame is fully tied. This may be restrictive in a tall building with a small plan area, where the curing period for the in-situ concrete infill is less than the erection time for a complete floor. In this case, bracing props are left in position for longer than usual so that the erection of the frame can proceed at a greater rate. ■

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PRECAST CONCRETE FRAME BUILDINGS

DESIGN GUIDE

APPENDIX

EXAMPLE SPECIFICATION
FOR STRUCTURAL PRECAST CONCRETE FRAMES

INTRODUCTION

■ ■ This Appendix gives an example performance specification for precast concrete frames. Where a composite precast concrete and steelwork frame is to be specified, an appropriate specification for the steelwork, such as that from the National Building Specification (NBS), should be added.

This Appendix covers the main parameters (see Section 2, Figure 2.1) which will need to be included when specifying precast concrete frames, but it has been written on the assumption that the client will appoint an architect and/or engineer with overall design responsibility for the building of which the frame forms an important part.

This Specification also defines the broader aspects of procuring the frame. This is to enable the architect and engineer to be satisfied that the design, production and erection of the precast concrete frame have been undertaken properly, and for them to integrate the design of the frame into the overall design of the building.

The Specification is intended to be used in conjunction with standard preliminaries and to be compatible with current NBS documents. To that end, the NBS section relating to precast concrete cladding has been used as a model.

Preliminaries

As an aide-mémoire for the specifier, the following is a list of the required preliminary clauses which will be found in the appropriate sections of the NBS. The list is not comprehensive, but covers those items of particular relevance and importance when precast concrete frames are being specified.

- **Site** Ascertain local conditions before tendering by visiting site and/or reference to client's site plans and other supplied documentation.
- **Obscurities** Resolve before tendering.
- **Contract without quantities** Allow for all necessary work.
- **Tender analysis** Submit in detail within a stated period upon request.
- **Pricing errors** Correction by Quantity Surveyor.
- **Attendances** To be defined.
- **Erection statement** Submit outline method and sequence of erection with tender.
- **Technical data** As applicable, to be submitted with tender.
- **Manufacturers and suppliers** Successful tenderer to provide list of all products where selection is by precaster.
- **Quality control** Provide statement with tender. Successful tenderer to describe procedures, implementation, staff, and responsibilities.
- **Sub-letting** General policy to be submitted with tender. Successful tenderer to submit names and addresses for all sub-let work and bought-in items.
- **BSI documents** Define current version.
- **Manufacturers' references** Current at time of tender.
- **Site data** Technical data, drawings, and specification to be available on site.
- **Person in charge** Provide details of Erection Supervisor prior to commencement of erection and give notice of any change.
- **Setting out** Precaster to check Main Contractor's setting out and obtain written approval to proceed.

Specification

The Specification falls broadly into six sections:

Structure	–	Item 1
General	–	Items 2 to 5
Design	–	Items 6 to 13
Concrete	–	Items 14 to 16
Production	–	Items 17 to 28
Erection	–	Items 29 to 38

The clauses which follow relate to a typical precast concrete structure which may consist of precast concrete columns, beams, spandrel panels, floors, stairs and structural walls, and structural steel purlins, trusses, beams columns and bracing.

This Specification requires values to be inserted in, or minor amendments made to, those clauses marked with an asterisk. In such cases, a comment on the modification is given beneath the clause. More detailed revisions or additional clauses may be necessary, depending on the nature of the project.

'CA' refers to the Contract Administrator. In all probability this will be the Architect, who will refer relevant matters to the Engineer.

It is intended that specification clauses for precast concrete frames should be included in NBS documents, but in the interim this specification may be adopted to form part of the tender documents. ■

EXAMPLE SPECIFICATION

■ ■ To be read with Preliminaries. The successful tenderer may be asked to submit more detailed information on certain clauses within a period of seven days.

TYPE OF STRUCTURE

1 Precast concrete structure

1.1* To comprise columns, beams, floors, spandrel panels, stairs and structural walls conforming with the requirements of the tender drawings.

This clause may need to be amended to suit the range of components expected in the project. For example, the design may not include spandrel panels.

GENERAL

2 Scope of work

2.1 The design, manufacture and erection of the precast concrete units referred to above including all site works necessary to produce a fully stable structure able to withstand all temporary and permanent, vertical and lateral loading at all stages of erection and on completion.

3 Information to be submitted with tender

3.1 General arrangement plans and sections at each floor level showing the layout and sizes of all precast concrete components together with the extent of all in-situ concrete provided.

3.2 A description of the construction including materials, method of erection and finishes. Any proposed departure from the tender documents must be clearly indicated.

3.3 A bar chart programme showing design, design approval, materials procurement, mould manufacture, production, erection and handover.

3.4 A full description of builder's work, special provisions and special attendances to be provided by others.

3.5 Limitations on cutting of holes in floor slabs, forming of chases and recesses and fixings to precast concrete components.

4 The contract sum analysis

4.1 A categorized breakdown of the precast concrete units and ancillary work should be given.

5 Programme constraints

5.1 Complete and handover to following trades in an agreed sequence. Construct stair flights and landings as main frame and floors proceed.

DESIGN AND PERFORMANCE REQUIREMENTS

6 Design and co-ordination

6.1 Design and detail the work and provide complete production information (e.g. fabrication and installation drawings, design calculations, and specifications) based on the drawings, this Specification and other information provided, liaising with the CA, Contractor and others as necessary to help ensure co-ordination of the work with related building elements and services and to integrate the design of the structure with the total building. Provide appropriate representation at regular design/progress meetings as required by the CA.

6.2 Request additional information as necessary from the CA and/or Contractor and provide information as necessary in time to meet the programme. Do not scale dimensions from drawings. Obtain from the CA any dimensions required but not given in figures on the drawings nor calculable therefrom. Report any discrepancies for clarification.

6.3* Submit or re-submit as appropriate .... copies of the GA drawings, ... copies of calculations, ... copies of unit details to the CA.

Insert appropriate number of copies of each item. Normally more for the GAs than for other details.

6.4 The CA will inspect the design/production information, record comments and return to the Precast Frame Contractor. The CA will require seven days for such examination of design/production information.

6.5 Make any necessary amendments in accordance with any comments of the CA and without delay. Unless and until it is confirmed that re-submission is not required, re-submit for further checking and comment, and incorporate any necessary further amendments all as before.

7 Basis of design

7.1* Design the precast concrete structure, its components and connections in accordance with all relevant current British Standards and to satisfy the requirements of the relevant Building Regulations.

If specific documents are to form the basis of design they should be specified in this section. This will often be necessary in order to qualify the subsequent clauses, for example: design precast concrete in accordance with BS 8110; design structural steelwork in accordance with BS 5950.

7.2* Design the precast concrete structure as a stable structure able to withstand all temporary and permanent, vertical and lateral loading without additional framing or stiffening from cladding, block walls, partitions, roof bracing, etc. Accept responsibility for the stability and structural integrity of the precast concrete structure during erection and support as necessary.

Amend text to include any restrictions on the use or location of any stiffening elements, bracing, etc.

7.3* Design precast concrete structure to withstand the effects of accidental damage and to satisfy relevant Building Regulations.

Specify documents in text if they are to be different from Clause 7.1.

7.4* Design column bases as pinned (or fixed).

Amend to suit foundation design, for example, restrictions on any base restraint or column/foundation connection.

8 Loading criteria

8.1* Design the structure for the following characteristic dead and imposed loads.

Specify documents in text, e.g. BS 6399

8.2* Wind loading

Basic wind speed	..... m/s
Topography factor, S1	.....
Ground roughness category	.....
S2, from table 3	.....
Statistical permanent factor, S3	.....
Statistical temporary factor, S3	.....

Insert appropriate values so as to provide consistency in tender.

8.3* Imposed floor loading

All general floor areas	..... kN/m^2
Plant room floor	..... kN/m^2

8.4* Additional loading

Partitions	..... kN/m^2
Ceilings	..... kN/m^2
Suspended services	..... kN/m^2
Floor finishes	..... kN/m^2

Other loads as shown on drawings supplied, for example, isolated plant items.

8.5* Roof loading

Dead load	..... kN/m^2 on plan
Imposed load	..... kN/m^2 on plan

Insert values in 8.3 to 8.5 and extend to cover any variations at different levels or for specific locations.

9 Deflections

9.1* Limit vertical deflections under imposed load to L/..... or ..... mm whichever is the lesser, where L is the span of the beam or slab. Limit lateral deflection in any one storey under wind load to H/..... where H is the floor-to-floor storey height.

This clause need only be included and specific deflection criteria given where different from those contained in the documents listed in Clause 7.1.

10 Fire resistance

10.1* The structure is to be designed for a fire resistance of ... hour(s).

Insert appropriate value and add any other specific requirements in respect to defined areas or parts of the building.

11 Integrity

11.1 The precast concrete components must be reinforced as necessary to allow for the effects of service loads, handling stresses, creep, shrinkage and deflection.

12 Calculations

12.1 Provide design calculations and details together with any relevant test data to enable design assumptions to be verified. Such calculations, details and data are also to be sufficient to satisfy the requirements of the Building Control Authority. Notwithstanding the above, the Frame Contractor is to remain responsible for the design and construction of the structure of the building insofar as it is designed and constructed by them. Assistance in the checking of drawings and calculations will be given by the Engineer but any failure to detect inconsistencies or errors will not relieve the Precast Frame Contractor of responsibility.

13 Durability

13.1* Subject to other specified requirements, the quality of concrete and nominal cover to reinforcement (including secondary reinforcement) must be related to the following environment: ....

To ensure design compatibility between tenders insert the design environment for elements of the building and, if not adequately defined in the specified documents, any requirements for aggregate type, grade of concrete, cement content, nominal cover, etc.

13.2 Increase the nominal cover to allow for depth of exposure on exposed aggregate finishes.

13.3 If galvanized or stainless steel reinforcement is to be used submit proposals for grade of concrete and nominal cover for approval.

MAKING CONCRETE

14 Concrete mixes generally

14.1 Constituent materials, composition of mixes, production of concrete, information to be provided, sampling, testing and compliance: all to be in accordance with BS 5328 unless otherwise specified.

14.2 The mix constituents to comply with the limitations given in BS 8110 in respect to the use and location of the elements.

14.3 Rate of sampling for compressive strength testing: not less than one sample for each day of use.

15 Correlated records

15.1 Correlated records must be maintained for each mix type, including:
- Composition of the mix, including any admixtures.
- Results of slump and other tests carried out at the casting yard.
- Laboratory test reports, including cube identification numbers.

16 Test laboratory

16.1 All compressive strength testing to be carried out by one laboratory, either NATLAS Accredited or other approved. Submit the name of the selected laboratory to CA as soon as possible and in any case before commencing manufacture.

PRODUCTION

17 Generally

17.1 Provide evidence of satisfactory quality control procedures and their effective implementation. Allow access by the CA or nominated representative to all manufacturing facilities during working hours subject to 48 hours notice in writing.

18 Moulds

18.1 Moulds must be so constructed that casting deviations can be controlled to give compliance with clause 21.1. Construct to prevent loss of grout. Design to permit demoulding without damage to the units. Coat evenly with a suitable release agent. Maintain in clean, sound condition and inspect carefully for defects before each re-use.

19 Plain smooth finish

19.1* Produce an even finish with a sheet material (e.g. plywood).

19.2* Abrupt irregularities to be not greater than 5 mm. Gradual irregularities, expressed as maximum permissible deviation from a 1 m straight edge, to be not greater than 5 mm.

19.3* Blowholes less than 10 mm in diameter will be permitted but otherwise surface to be free from voids, honeycombing, segregation and other large defects.

19.4* Making good: projecting fins are to be removed and rubbed down with a carborundum stone but otherwise the finish is to be left as struck.

Modify 19.1 to 19.4 if necessary to reflect class and type of finish given in BS 8110 or further define for quality of surface.

20 Smooth floated finish

20.1* Use a hand float, skip or power float to give an even surface with minimal ridges or steps.

Amend as necessary where particular finish is required.

21 Manufacturing accuracy

21.1* Finished dimensions of precast concrete components to comply with the manufacturing tolerances given in Tables 1 and 2 of BS 5606, except where qualified as follows: ....

Specification for dimensional deviations on length, height, thickness, etc need particular consideration (see Section 3).

21.2 Check the overall dimensions, straightness, squareness, twist and flatness of the mould(s) immediately before each re-use, and of each component as soon as possible after demoulding. Make adjustments to moulds as necessary.

22 Camber

22.1* Reinforced components are not to be cambered unless agreed in advance with the CA. For prestressed concrete components, the predicted camber in accordance with Clause 6.11.4 of BS 8110:Part 1 shall not be greater than 25 mm unless otherwise agreed.

22.2* Variation in camber between adjacent floor units of the same span and loading should, except where otherwise qualified, not exceed half the predicted value.

Amend values in 22.1 and 22.2 if necessary and include qualifications for any particular units.

23 Fixings

23.1 Lifting and adjustment devices which are away from the external face and edges and recessed for subsequent filling with mortar may be of galvanized steel. Other devices must be of a suitable type of stainless steel or non-ferrous metal. Prevent bimetallic corrosion and staining.

24 Reinforcement

24.1 Unless otherwise specified:
- Reinforcement to BS 4449 and/or BS 4483, cut and bent to BS 4466.
- Prestressing strand to BS 5896.

All reinforcement to be obtained from a CARES-registered supplier.

24.2 Galvanized reinforcement: galvanized to BS 729 after cutting, chromate treated.

24.3 Stainless steel reinforcement: to BS 6744, type 304 S31 or alternative to CA's approval.

24.4 Surface condition: to Clause 7.4 of BS 8110 at time of placing.

24.5 Fix accurately and securely using tying wire, approved steel clips, or tack welding to BS 7123 if permitted. Wire or clips must not encroach into the concrete cover greater than their thickness or diameter.

25 Casting and curing

25.1 Thoroughly compact concrete by mechanical vibration.

25.2 Do not demould components prematurely.

25.3 Prevent damage to and distortion of immature components from movement, vibration, overloading, physical shock, rapid cooling and thermal shock.

25.4 Do not deliver components to site without prior approval, until at least three days after casting.

26 Records

26.1 Keep complete records for each component including the following information:
- An identification number.
- Correlation with records of mixes.
- Date of manufacture.
- Dates and results of all tests, checks and inspections.
- Dimensions related to specified levels of accuracy.

26.2 Records to be available for inspection on request.

27 Identification

27.1 Mark all components in stock with contract reference.

ERECTION, FIXING, JOINTING

28 Erection, fixing, jointing

28.1 To be carried out by the Contractor who must:
- Provide clear and comprehensive instructions and ensure that they are understood by the site operatives.
- Provide adequate site supervision by suitably skilled person(s).

29 Protection

29.1 Prevent overstressing of components during transit, handling, storage and fixing. Lift components at designed lifting points only, using special lifting devices and cradles as necessary.

29.2 Store components on level bearers clear of the ground and separate with spacers at designed locations.

29.3 Prevent damage to components.

30 Prestressed floor slabs

30.1 Inspect on arrival on site and prior to grouting or in-situ concreting. Do not erect slabs which exhibit any of the following:
- Excessive transport or handling damage.
- Manufacturing defects.
- Excessive cracks.
- Excessive areas of lean or honeycombed concrete.
- Excessive camber.
- Tendon 'pull-in' exceeding 5 mm.

31 Holding-down bolts

31.1 Supply holding-down bolts to the Main Contractor to suit programme requirements. Make allowance for holding down bolts positioned within an accuracy of ± 6 mm by the Main Contractor.

32 Accuracy of erection

32.1 Construct the works to levels of accuracy as given in Table 1 of BS 5606

Include here any specific or more onerous requirements than provided for by BS 5606.

Do not rectify work which fails to meet the specified levels of accuracy without approval.

33 Foundations

33.1 Before starting erection, inspect the foundations including holding down bolts and dowel pockets for line and level, and confirm accuracy in writing to the CA.

34 In-situ concrete

34.1 Where in-situ concrete is designed into the structure, provide details of mixes and production. Make cubes for each day of concreting, test and submit to the CA.

35 Grout

35.1 Provide details of the mix and production. Make cubes as required, test and submit written reports to the CA.

36 Damaged units

36.1 Do not repair without approval. Such approval will not be given where the components are badly damaged or where the proposed repair would impair performance. Immediately remove rejected units from the site.

37 Site welding

37.1 All site fillet welding must be in accordance with BCSA National Structural Steelwork Specification for Building Construction or BS 7123 as appropriate. Take due account of the embedment of steel fittings in concrete where this will affect welding. Site welds must be inspected, visually 100% and MPI 10% as a minimum requirement. All welds failing visual inspection must be MPI tested. All welding defects must be rectified prior to work being covered up, and inspection records must be available as required. Procedures for butt welding to be submitted to the CA for approval.

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624.012.3:624.072.33

PRECAST CONCRETE FRAME BUILDINGS

DESIGN GUIDE

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British Cement Association Publication 47.024

BETTER BUILT IN
concrete