CHAPTER 1
Precast Concepts, History and Design Philosophy

The background to the relevance of precast concrete as a modern construction method for multi-storey buildings is described. The design method is summarised.

1.1   A Historical Note on the Development of Precast Frames

Precast concrete is not a new idea. William H. Lascelles (1832–85) of Exeter, England devised a system of precasting concrete wall panels, 3 ft × 2 ft × 1 inch thick, strengthened by forged, ⅛ inch-square iron bars. The cost was 3d (£0.01) per ft². Afterwards, the notion of 'pre-casting' concrete for major structural purposes began in the late nineteenth century, when its most obvious application – to span over areas with difficult access – began with the use of flooring joists. François Hennebique (1842–1921) first introduced precast concrete into a cast-*in situ* flour mill in France, where the self-weight of the prefabricated units was limited to the lifting capacity of two strong men! White [1.1] and Morris [1.2] give good historical accounts of these early developments.

The first precast and reinforced concrete (rc) frame in Britain was Weaver's Mill in Swansea. In referring to the photograph of the building, shown in Figure 1.1, a historical note states: . . . *the large building was part of the flour mill complex of Weaver and Co. The firm established themselves at the North Dock basin in 1895–6, and caused the large ferro-concrete mill to be built in 1897–98. It was constructed on the system devised by a Frenchman, F. Hennebique, the local architect being H. C. Portsmouth . . .* At this time Louis Gustave Mouchel (1852–1908, founder of the Mouchel Group) was chosen to be Hennebique's UK agent. Mouchel used a mix of cast-*in situ* and prefabricated concrete for a range of concrete framed structures, building at the rate of 10 per year for the next 12 years.

The structure was a beam-and-column skeletal frame, generally of seven storeys in height, with floor and beams spans of about 20 feet. The building has since been demolished owing to changes in land utilisation, but as a major precast and reinforced concrete construction it pre-dates the majority of early precast frames by about 40 years.

Bachmann and Steinle [1.3] note that the first trials in structural precast components took place around 1900, for example at Coignet's casino building in Biarritz in 1891, and Hennebique and Züblin's signalman's lodge in 1896, a complete three-dimensional cellular structure weighing about 11 tons [1.3].

During the First World War storehouses for various military purposes were prefabricated using rc walls and shells. Later, the 1930s saw expansions by companies such as Bison, Trent Concrete and

Figure 1.1   Weaver's Mill, Swansea – the first precast concrete skeletal frame in the United Kingdom, constructed in 1897–98 (courtesy of Swansea City Archives).

Girling, with establishments positioned close to aggregate reserves in the Thames and Trent Valley basins. The reason why precast concrete came into being in the first place varies from country to country. One of the main reasons was that availability of structural timber became more limited. Some countries, notably the Soviet Union, Scandinavia and others in northern Continental Europe, which together possess more than one-third of the world's timber resources but experience long and cold winters, regarded its development as a major part of their indigenous national economy. Structural steelwork was not a major competitor at the time outside the United States, since it was batch-processed and thus relatively more expensive.

During the next 25 years developments in precast frame systems, prestressed concrete (psc) long-span rafters (up to 70 feet), and precast cladding increased the precasters' market share to around 15 per cent in the industrial, commercial and domestic sectors. Influential articles in such journals as the *Engineering News Record* encouraged some companies to begin producing prestressed floor slabs, and in order to provide a comprehensive service by which to market the floors these companies diversified into frames. In 1960 the number of precast companies manufacturing major structural components in Britain was around thirty. Today it is about eight.

Early structural systems were rather cumbersome compared with the slim-line components used in modern construction. Structural zones of up to 36 inches, giving rise to span/depth ratios of less than 9, were used in favour of more optimised precasting techniques and designs. This could have been called the 'heavy' period, as shown in C. Glover's now classic handbook *Structural Precast Concrete* [1.4]. Some of the concepts shown by Glover are still practised today and one cannot resist the thought that the new generation of precast concrete designers should take heed of books such as this. It is also difficult to avoid making comparisons with the 'lighter' precast period that was to follow in the 1980s, when the saving on total building height could, in some instances, be as much as 100 to 150 mm per floor.

Attempts to standardise precast building systems in Britain led to the development of the National Building Frame (NBF) and, later, the Public Building Frame (PBF). The real initiative in developing these systems was entrenched more in central policy from the then Ministry of Public Building and Works than by the precasting engineers of the building industry. The NBF was designed to provide: . . . *a flexible and economical system of standardised concrete framing for buildings up to six*

storeys in height. It comprises a small number of different precast components produced from a few standard moulds [1.5].

The consumer for the PBF was the Department of Environment, for use within the public sector's expanding building programme of the 1960s. Unlike the NBF, which was controlled by licence, the PBF was available without patent restrictions to any designer. The structural models were simple and economical: simply supported, long-span, prestressed concrete slabs up to 20 inches deep were half recessed into beams of equal depth. By controlling the main variables, such as loading ($3+1\,kN/m^2$ superimposed was used throughout), concrete strength and reinforcement quantities, limiting spans were computed against structural floor depths. Figures 1.2 and 1.3 show some of the details of these frames. Diamant [1.6] records the international development of industrialised buildings between the early 1950s and 1964. During this period the authoritative Eastern European work by Mokk [1.7] was translated into English, and with it the documentation of precast concrete had begun.

Unfortunately, the modular design philosophy was reflected in the façade architecture. The results were predictable, exemplified at Highbury Technical College in Portsmouth (now a part of the University of Portsmouth) shown in Figure 1.4. The precast industry has found it difficult to dispel the legacy of such architectural brutalism.
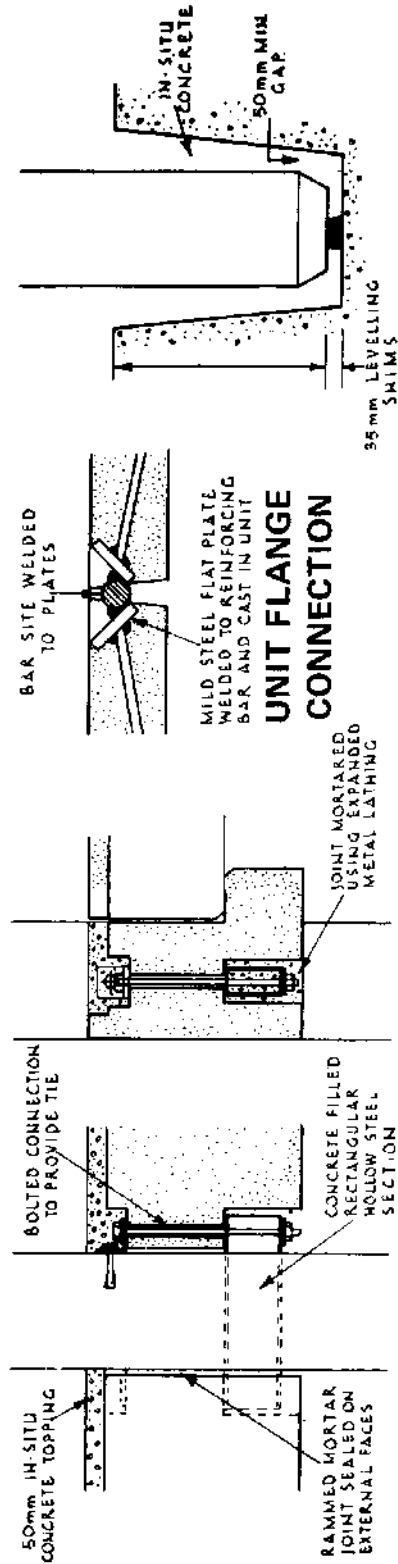
Following the demise of the NBF and PBF, precast frame design evolved towards more of a client-based concept. Standard frame systems gave way to the incorporation of standardised components into bespoke solutions. The result, shown in Figure 1.5 of Western House (1990), Surrey Docks (1990) and Merchant's House (1991), established the route to the versatile precast concrete concepts of the present day.

In the mid-1980s, the enormous demands on the British construction industry led developers to look elsewhere for building products, as the demands on the British precast industry far exceeded its capacity. Individual frame and cladding companies (with annual turnovers of between £1 m and £3 m) were being asked to tender for projects that were singularly equal to or greater in value than their annual turnovers. Programmes were unreasonably tight and it seemed that the lessons learned from mass-market-led production techniques of the 1960s had gone unheeded. One solution was to turn to Northern Europe, where the larger structural concrete prefabricators were able to cope with these demands. Concrete prefabricated in Belgium was duly transported to the London Docklands project, shown in Figure 1.6.

In making a comparison of developments in Europe and North America, Nilson [1.8] states: *Over the past 30 years, developments of prestressed concrete in Europe and in the United States have taken place along quite different lines. In Europe, where the ratio of labor cost to material cost has been relatively low, innovative one-of-a-kind projects were economically feasible. . . . In the U.S. the demand for skilled on-site building labor often exceeded the supply, so economic conditions favored the greatest possible standardisation of construction . . .*
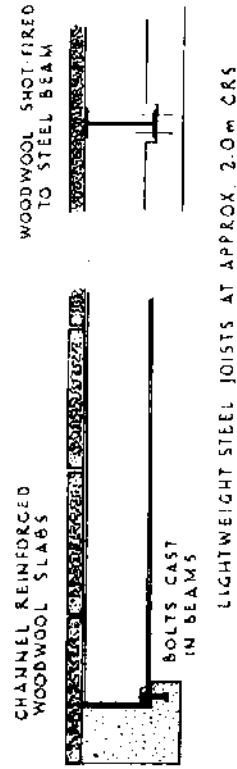
North America's production capabilities are an order of magnitude greater than those of Europe. Figure 1.7 shows the construction of a 30-storey, 5000-room hotel and leisure complex in Las Vegas. The conditions to which Nilson refers are changing. The gap between labour and material costs in Europe is now closer to that of North America. At the same time, progressively lower oil and transportation costs into the early twenty-first century made it feasible to manufacture components virtually anywhere in the world and transport them to regions of high construction demand. Recent indications are that increasingly scarce energy resources and narrowing pay differentials will reverse this trend.

While the market share for complete precast concrete frames has remained constant in the UK, the development of high-strength concrete for columns and the use of innovative shallow prestressed concrete beams, together with speed of construction to rival that of steelwork, has led to successful tower buildings in Northern Europe, particularly in Belgium and Holland. The twin-tower building 'galaxy' in Brussels, Figure 1.8, is such an example. With column sizes of 600 mm diameter (cast in two-storey heights using $95\,N/mm^2$ concrete strength) and beam and floor spans up to 9.2 m × 405 mm depth, construction rates achieved 2 storeys in 8 working days [1.9].

COLUMN FOUNDATION

IN-SITU CONCRETE

50mm MIN GAP

35mm LEVELLING SHIMS

TYPICAL STRUCTURAL DETAILS

UNIT FLANGE CONNECTION

BAR SITE WELDED TO PLATES

MILD STEEL FLAT PLATE WELDED TO REINFORCING BAR AND CAST IN UNIT

BEAM TO COLUMN CONNECTION

JOINT MORTARED USING EXPANDED METAL LATHING

BOLTED CONNECTION TO PROVIDE TIE

CONCRETE FILLED RECTANGULAR HOLLOW STEEL SECTION

50mm IN-SITU CONCRETE TOPPING

RAMMED MORTAR JOINT SEALED ON EXTERNAL FACES

ALTERNATIVE LIGHTWEIGHT ROOF

WOODWOOL SHOT-FIRED TO STEEL BEAM

CHANNEL REINFORCED WOODWOOL SLABS

LIGHTWEIGHT STEEL JOISTS AT APPROX. 2.0m CRS

BOLTS CAST IN BEAMS

Figure 1.2 Typical structural details for the National Building Frame [1.4].
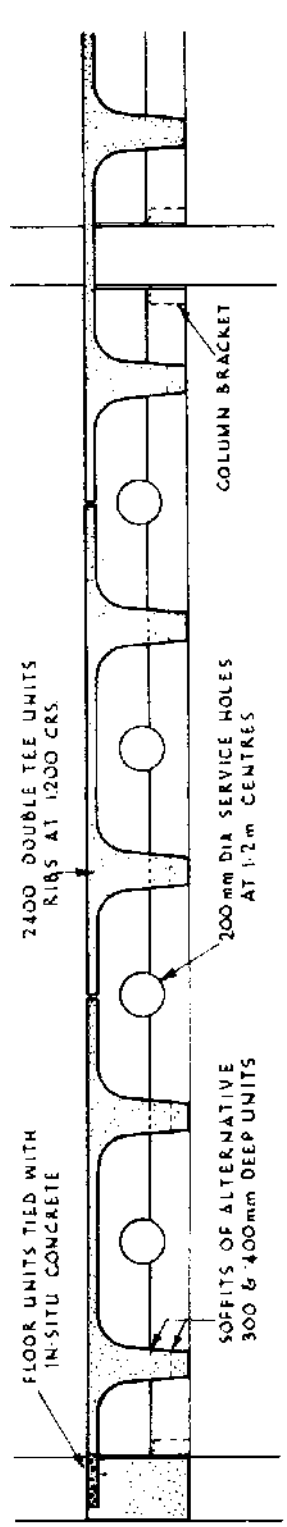
TRANSVERSE SECTION THROUGH FLOOR OR ROOF

FLOOR UNITS TIED WITH
IN-SITU CONCRETE

2400 DOUBLE TEE UNITS
RIBS AT 1200 CRS.

200 mm DIA SERVICE HOLES
AT 1·2m CENTRES

COLUMN BRACKET

SOFFITS OF ALTERNATIVE
300 & 400mm DEEP UNITS
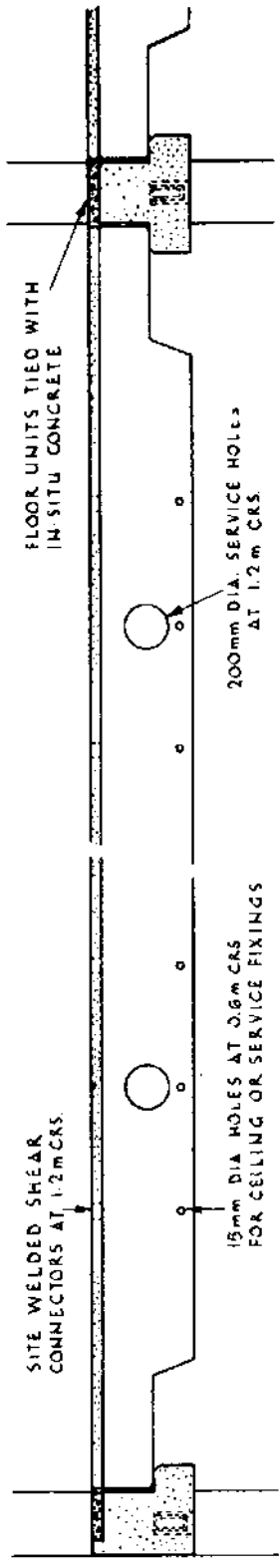
LONGITUDINAL SECTION THROUGH FLOOR

FLOOR UNITS TIED WITH
IN-SITU CONCRETE

200mm DIA. SERVICE HOLES
AT 1·2 m CRS.

SITE WELDED SHEAR
CONNECTORS AT 1·2m CRS.

15mm DIA HOLES AT 0.6m CRS
FOR CEILING OR SERVICE FIXINGS

Figure 1.3 Floors used in the National Building Frame [1.4].

Figure 1.4 Precast construction of the 1960s using the National Building Frame, The building is Highbury College, now part of the University of Portsmouth (courtesy of Costain Building Products).

(a)

Figure 1.5 Examples of precast construction of the 1980s. (a) Western House, Swindon (courtesy Trent Concrete Ltd); (b) Surrey Docks, London (courtesy Crendon Structures); (c) Merchant House, London [1.10].

(b)

(c)

Figure 1.5   (*Continued*)

Changes to the way in which the construction industry should operate in a 'zero-waste and zero-defects' environment were given in the 1998 Egan Report [1.11]. The report called for sustained improvement targets: 10 per cent in capital construction costs; 10 per cent in construction time; 20 per cent of defects, and a 20 per cent increase in predictability. Further, the report goes on: . . . *The industry must design projects for ease of construction, making maximum use of standard components and processes.* Although the reports did not use the term 'prefabrication', to many people that is what 'predictability' and 'standard components' mean. The precast concrete industry is ideally placed to accommodate these higher demands by using experienced design teams and skilled labour in a quality-controlled environment to produce high-specification components. Figure 1.9 illustrates this in the repetitive use of granite cast spandrel beams and columns to form a building in the convoluted shape of a shell. Since 2000, high-quality architectural finishes have been more widely adopted for the exposed structural components, as illustrated in the integrated structure in Figure 1.10. The requirement for off-site fabrication will continue to increase as the rapid growth in management contracting, with its desire for reduced on-site processes and high-quality workmanship, will favour controlled prefabrication methods. The past five years have witnessed extensive developments in student accommodation – for example in Figure 1.11, where some 600 rooms were constructed using precast wall and floor systems in just over eight months, and were completed for occupancy within 18 months of site possession in 2009.

(a)

(b)

Figure 1.6 An early example of quality architecturally detailed precast concrete imported from Belgium. (a) Overall impression. (b) Architectural detail. (Courtesy of A. Van Acker, TU Ghent.)

Figure 1.7 MGM Hotel and Casino at Las Vegas, US constructed in 1992 (courtesy of A. T. Curd).

Figure 1.8 36-storey precast skeletal tower buildings in Brussels (courtesy of Ergon, Belgium).

Figure 1.9 Granite aggregates in spandrel beams and columns, polished to reflect the Australian sunshine at No. 1 Spring Street, Melbourne, 1990.

(a)   (b)

Figure 1.10 Asticus Building, London, 2010. Precast cruciforms form (a) the exterior structure, and (b) architectural finish.

Figure 1.11   Student accommodation buildings at the University of West of England, Bristol. Precasted by Buchan, UK.

1.2   The Scope for Prefabricated Buildings

1.2.1   Modularisation and standardisation

The precast industry is still struggling to overcome the misconceptions of modular precast concrete buildings. This is not surprising, as many texts refer to: . . . *the design of a precast concrete structure on a modular grid. The grid should preferably have a basic module of 0.6 m* . . . [1.12]. The Continental Europeans introduced the phrase 'modular coordination', which meant the interdependent arrangement of dimensions, based on a primary value accepted as a module [1.13]. This dimension was 30 cm horizontally and 10 cm vertically. Moreover, the storey height in precast concrete apartment buildings was fixed at 280 cm, with the horizontal grid dimension on a 30 cm incremental scale between 270 cm and 540 cm. Strict observance of these rules facilitated the optimum assembly of prefabricated structures – in other words, all prefabricated buildings looked the same. And they were nearly all boxes – 'People are getting tired of this shoe-box architecture', said Harry Seidler, OBE, architect for the nautilus-shaped building in Figure 1.9.

There is a clear distinction between 'modular coordination' and 'standardisation'. The precast industry deplores the former and encourages the latter. What is the difference and how can this be?

Modularisation offers zero flexibility off the modular grid. The end product is evident in the comparison of the two buildings adjacent to Vauxhall Bridge in London, shown in Figure 1.12. Interior architectural freedom is possible only in the adoption of module quantities and configuration, and one cannot escape the geometrical dominance and lack of individuality of the older building on the left of the photograph. Exterior façades may of course be varied indefinitely, as in the 'twin façade'

Figure 1.12   Examples of past and present use of precast concrete, Vauxhall Bridge, London.

system shown in Figure 1.13, but that requires a full precast perimeter wall. In skeletal frames; one need go no further than the adoption of families of modular precast concrete components to obtain the optimum solution for any building, within reasonable limits.

Industrial modularised buildings were introduced in Europe in the 1950s, during the mass construction period following the Second World War. The problems in the architectural and social environment brought a re-emergence of traditional methods, and closer control on design and factory production. This has inevitably led to a new philosophy in what is called the 'modulated hierarchical building system' [1.14], which aims at the subdivision of a building into:

- functional systems, i.e. space utilisation both vertically and horizontally, personnel coordination, adaptability to changing needs
- technical systems, i.e. the structural design, the façade, mechanical and electrical services, waste disposal, and air-conditioning systems.

Precast modular frame manufacturers have been able to synthesise these requirements through continuous development of improved products and creative use of limited ranges of precast concrete products.

Standardisation is quite different from modularisation. It refers to the manner in which a set of predetermined components are used and connected. The buildings shown in Figures 1.14 to 1.16 were constructed using more or less the same family of standardised components. (The Reinforced Concrete Council has published case studies on precast frames, e.g. [1.10], where this may also be appreciated.) By adjusting beam depths, column lengths, wall positions, etc. the same components in any of these buildings could have been used to make a completely different structure. This is not possible with the modular system vision of the 1960s.

The four basic types of precast concrete structure are:

(1) The portal frame, Figure 1.17, consisting of columns and roof rafters or beams, provides large and adaptable ground-floor space. Portal frame structures are used for single-storey retail, warehousing, and industrial manufacturing facilities.
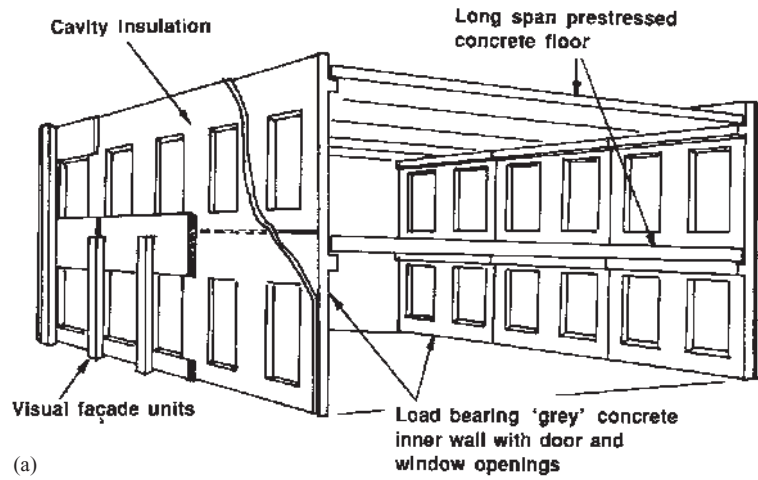
(a)

(b)

Figure 1.13 Principle of the twin facade system: (a) diagrammatic representation and (b) offices, Brussels.
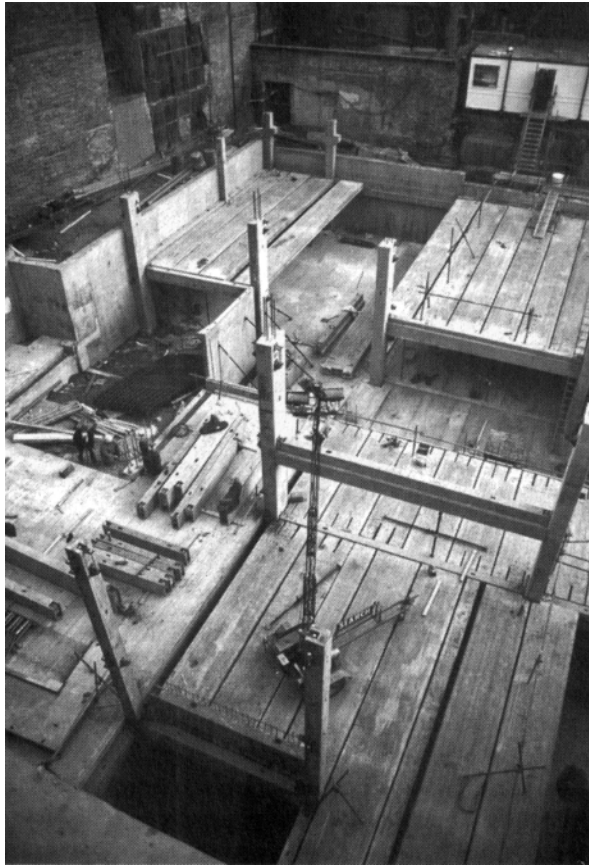
Figure 1.14 Structural 'grey' precast frame at Nottingham, UK (1995).

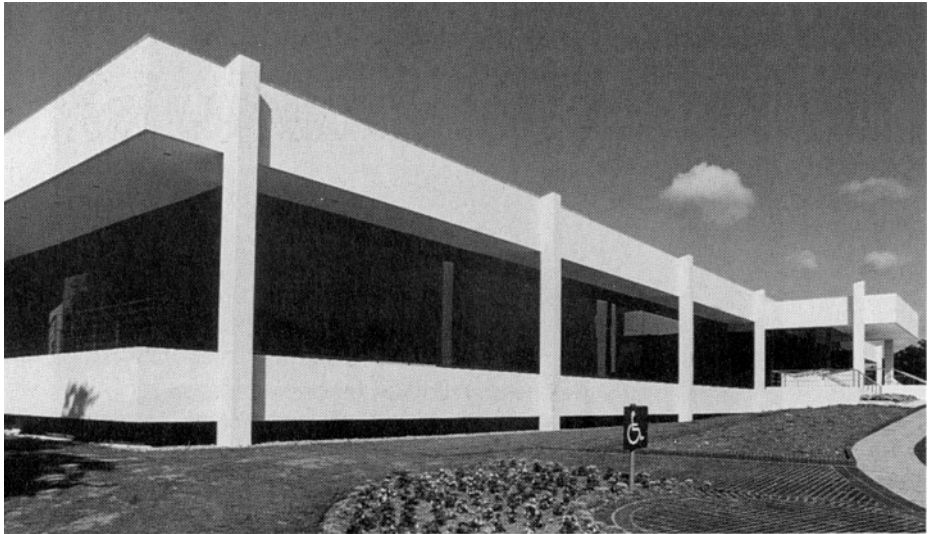**Figure 1.15** Commercial office development (courtesy of Crendon Structure, 1988).

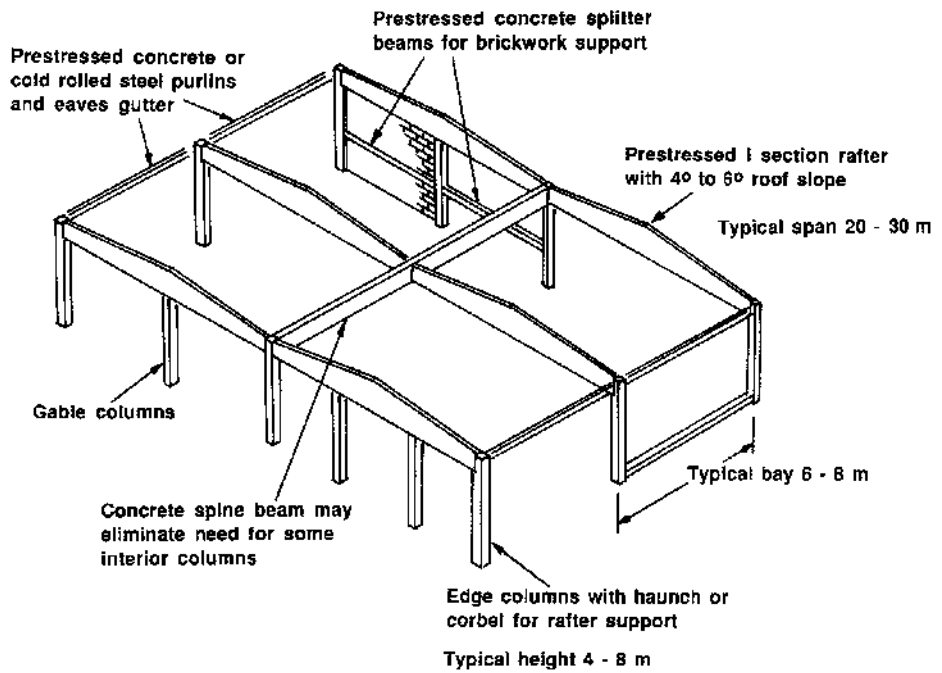**Figure 1.16**   Visual structural units with a polished concrete finish (courtesy of Trent Concrete Ltd, 1992).

Figure 1.17   Typical arrangement of portal frames.

Figure 1.18   Wall frames used in apartment buildings and retail units near Sydney.

Figure 1.19   Crosswall construction.

(2) The crosswall frame, Figures 1.18 and 1.19, consisting of solid or voided vertical wall and horizontal slab units only. Here the structural walls serve as shear walls to resist lateral forces that increase with height, and also as acoustic and thermal partitioning. However, the walls interrupt the internal space and reduce functional flexibility. Wall frame structures are used extensively for multi-storey hotels, retail units, hospitals, housing, and partitioned offices.

(3) Volumetric, or cellular, structures such as that in Figure 1.20 are developments of wall frames in which a number of the walls and floors are constructed together as units. These units are suitable for high levels of factory installation of finishes and services, e.g. complete bathrooms, plumbing

Figure 1.20 Volumetric structure.

and wiring. Cellular units are then assembled to provide grouped, individual facilities such as student accommodation, hotel rooms or prisons.

(4) Skeletal structures, Figure 1.21, consisting of columns, beams and slabs for low- to medium-rise buildings, with a small number of walls for high-rise. Skeletal frames are used chiefly for commercial offices and car parks, where both clear spans and multiple storeys are required.

Skeletal frames are the most architecturally and structurally demanding, because in both disciplines designers feel that they have free rein to exploit the structural system by creating large, uninterrupted spans while reducing structural depths and the extent of the bracing elements. This results in a large proportion of the connections being highly stressed and difficult to analyse.

Non-structural or structural load-bearing panels or cells may also be incorporated in the frame, usually in the perimeter, and in the service cores. These units may be used as decorative cladding with a very wide range of finishes, textures, etc., thermal and sound insulation, fixings and other provisions. As decorative cladding, they are beyond the scope of this book and are documented elsewhere [1.15–1.17].

The skeletal structure is distinguished from other types because imposed gravity loads are carried to the foundations by beams and columns, and horizontal loads by columns and/or walls. The structural efficiency of a building is related, to a certain degree although not absolutely, to the mass of the structure. The volume of vertical structural concrete in a skeletal precast structure is in the order of 1 to 2 per cent of the volume of the building (for typical 3 to 3.5 m storey heights and 6 to 8 m beam spans). For a cross-wall frame the figure is closer to 4 per cent. Flooring is the most significant factor, with prestressed concrete voided slabs adding a further 4 per cent, compared with 7 to 8 per cent for solid concrete slabs.

1.3   Current Attitudes towards Precast Concrete Structures

The use of multi-storey precast concrete frames has evolved to improve the economy of high-specification buildings. Architectural precast concrete components are being used structurally on an
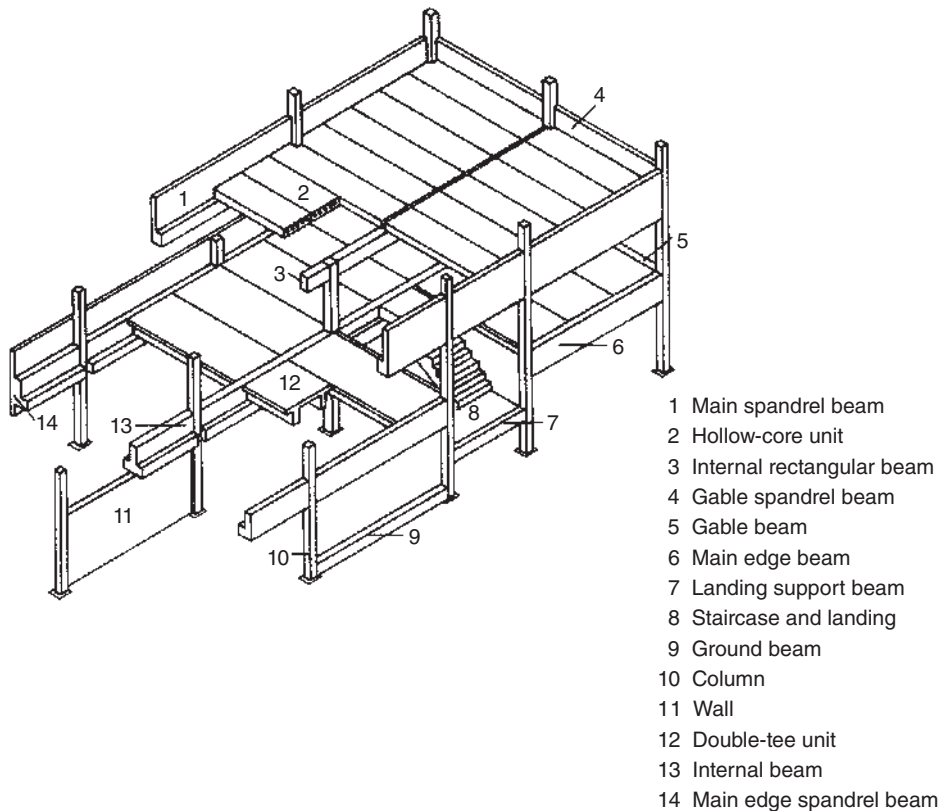
1 Main spandrel beam
2 Hollow-core unit
3 Internal rectangular beam
4 Gable spandrel beam
5 Gable beam
6 Main edge beam
7 Landing support beam
8 Staircase and landing
9 Ground beam
10 Column
11 Wall
12 Double-tee unit
13 Internal beam
14 Main edge spandrel beam

Figure 1.21   Skeletal structure and definition of components.

increasing number of prestigious commercial buildings, incorporating steelwork, timber and masonry for overall benefit. Designers are becoming more aware of the high-quality finishes possible in pre-fabricated units, affecting the way that the traditional precast structures are conceived and designed. The clients, and other parts of the construction industry, are calling for multi-functional design, where the use of all the components forming the building must be optimised.

The market share of commercial buildings, offices, schools, hotels, hospitals and car parks, etc. has fluctuated between as little as 5 per cent and nearly 15 per cent. Figure 1.22 shows the typical break-down of market share in the multi-storey building market. Figure 1.23 shows the approximate break-down of the UK's 2007 £2.6bn precast concrete market by product classification [1.18].

Compared with Continental Europe, Scandinavia and North America, the United Kingdom's precast concrete marketplace is small. Precast concrete frames, flooring and cladding represent approximately 22 per cent, 57 per cent, and 21 per cent of the total precast industry turnover, respectively.

To place this into context, Table 1.1 shows the approximate quantities of precast concrete flooring used in Europe in 1990 [1.19]. Total structural precast turnover in each country is nearly proportional. This variation in market penetration is not apparent in design and production matters, where concrete technology and production engineering are found to be broadly similar [1.20].

Use of precast concrete construction beyond five storeys has been rare in the UK until recently. Unlike the attitude in Continental Europe, North America and New Zealand, precast concrete is still considered as an alternative means of medium-rise (five to ten storeys) building construction to *in situ* concrete and structural steel. The possible reasons for this are three-fold:
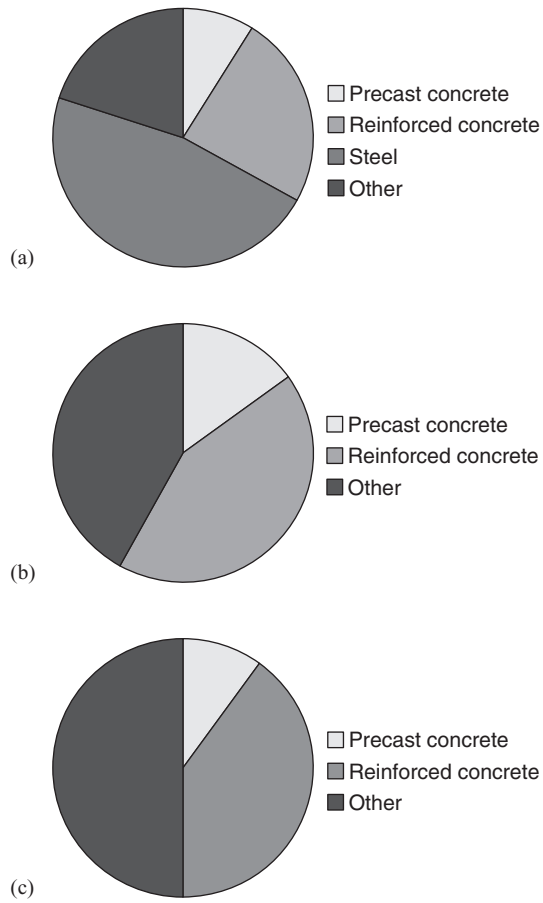
(a)

(b)

(c)

Figure 1.22   Market share for precast concrete in the multi-storey building market [1.18]. (a) Market share by value. (b) Non-housing market share. (c) Housing market share.
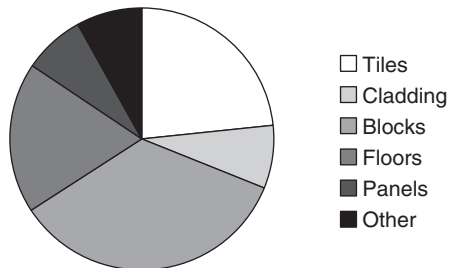

Figure 1.23   Market share for components of precast concrete.

Table 1.1   Quantities of precast concrete flooring used in Europe

Country	Production of hollow-cored flooring	
	millions of m^2	m^2 per capita
Germany	0.6	0.01
France	1.6	0.03
UK	2.8	0.05
Italy	3.4	0.06
Belgium	0.8	0.08
Denmark	0.7	0.15
Norway	0.9	0.22
Sweden	2.5	0.30
Netherlands	5.1	0.36
Finland	3.0	0.65

Note: On average 1 m^2 equates to about 0.12 m^3 of concrete, weighing 290 kg.

(1) There is a widespread lack of knowledge on the structural mechanics of precast construction, because the topic is rarely taught at a university and is not covered by post-graduate training in professional design practices. Precast has a low profile in academia, attracting little fundamental research funding from government. Few technical training courses have been organised, though the trade associations (The Concrete Centre, British Precast Concrete Federation, and Precast Flooring Federation) publish much general guidance and publicity material. As a result, precast concrete is perceived as difficult to specify for total building construction.

(2) Structural design and erection is nearly always (in 95 per cent of cases) carried out by the nominated precast manufacturer, thus depriving the consulting engineer of direct control over the frame design. Few clients, particularly government departments, will commit to a single-source manufacturer prior to design. There is a greater tradition of 'design and build' in the Continental construction industry.

(3) There is a lack of manufacturing capacity. Large-scale precast production demands capital investment. In a volatile economy, as has been seen in the UK since 1970, manufacturing companies have been forced to ride the crests and troughs with equal acumen if production capacities are to be maintained. Whereas structural steel can be stockpiled ready for cutting, and *in situ* concrete be poured at short notice, precast products must be manufactured to order.

In some countries precast concrete is viewed as a 'value-added product' and therefore carries a VAT surcharge over cast-*in situ* concrete.

On a worldwide scale the scope for high-specification precast concrete in buildings is still very large. The trend seems to be that in places where contracts are controlled by the contractor, rather than by the architect or client, precast is used mainly for the horizontal components, such as prestressed floors. This is reflected in the increase in use of precast concrete floors in residential building ground floors, up from 5 per cent in 1970 to 70 per cent in 2005. It is a well-known fact that precast manufacturers make less money from making vertical components than horizontal ones.

Furthermore, in certain regions, e.g. in the Middle East and Far East, the demand is growing at a rate greater than the increase in available technology. The major restraint for precast in the Far East is the very competitive cost for cast-*in situ* concrete, where site labour costs are typically single percentage points of the European costs. Building development is moving at such a swift pace that the frame geometry is fixed before the precast manufacturer has a chance to submit a more economical layout.

Table 1.2 Profit margins for US companies

Decade	Annual turnover	Annual profit (approx)
1970s	<$ 3.0 m	1.1%
	>$ 3.0 m	4.6%
1980s	<$ 7.5 m	1.8%
	>$ 7.5 m	8.2%
1990s	<$ 15.0 m	1.2%
	>$ 15.0 m	6.6%

Precast breaks down conveniently into three markets:

- residential (mainly 2–4 storeys)   5–15%
- commercial (offices, hotels, retail supermarkets, factories)   50–70%
- services (car parks, hospitals, colleges, sports stadiums)   20–40%

Product breakdown based on volume of concrete produced can be grouped into four markets:

- architectural façades, structural and non-structural   20–30%
- walls (non-decorative)   5–30%*
- floor slabs   50–60%
- beams and columns   10–15%

Note that volume of concrete produced is possibly not the best indicator with which to assess the industry. For example, the cost and the effort expended in manufacturing floor slabs is considerably less than for an architectural panel of similar weight, in some cases by a factor exceeding 10.

There is no doubt that the key to a successful precast concrete business is size. Output capacity has had a significant influence on the survival of precast companies in building. Using the American market as a role model (although a similar but scaled-down trend was seen in the UK and Europe), the profit margins for US companies were as shown in Table 1.2 [1.21].

Over this same period the approximate volume of precast reinforced and prestressed concrete produced per capita increased from 0.015 m^3 to 0.025 m^3 by the mid-1980s, but has fallen back to about 0.015 m^3 today.

1.4   Recent Trends in Design, and a New Definition for Precast Concrete

Responding to the reduced market of the 1990s, the industry saw a new movement in the design of precast structures, and a new definition to the word 'design'. It no longer meant '$\omega L^2/8$'. In a shrinking market the design of structural concrete frames for prestigious commercial buildings came under closer scrutiny as architects, design engineers and contractors strived to find optimum economy, speed of erection and the highest specification for their projects. The construction industry required a multiple choice in the selection of building components, and it was likely that the increasing demand on the performance of these components would overtake the existing technology used in their manufacture and utilisation.

Building design therefore became a multi-functional process, where the optimum use of all components forming the building was maximised. Attention was directed towards the structural frame,

*The production of walls for wall frames varies widely in different countries.

which could no longer be considered as serving only a structural function, but must be harmonised with the requirements of the building in total. Combined architectural/structural precast concrete components were therefore used on an increasing number of prestigious commercial buildings, as 'specifiers' became more aware of the high-quality finish possible in prefabricated units. However, changes had to be made to the way those traditional precast concrete structures were conceived and designed. Despite the cutbacks in staffing levels, the precast concrete industry was ideally placed to accommodate these higher demands by using experienced design teams and skilled labour in a quality-controlled environment to produce high-specification components, which served all the structural, architectural and services functions simultaneously.

An excellent example of the use of architectural/structural components is shown in Figure 1.16 above. White marble was originally chosen as separate cladding material to the external façade of a plain, precast concrete frame, including the expressed external columns. During the conceptual design stage it was realised that a white marble façade would have poor weathering characteristics, so it was replaced with a white, polished concrete, manufactured using 10 mm size Spanish Dolomite aggregate and white Portland cement, wet-ground and polished to produce a 'marble' appearance. The external façade was cast simultaneously with the structural components to reduce the overall cross section and eliminate unsightly joints to the front of the columns and beams [1.22].

Figure 1.24 shows prefabricated piers faced with load-bearing bricks used to support precast concrete floor units, spanning 14 m × 3.6 m wide, expressed as a rippling concrete band of vaulted units.
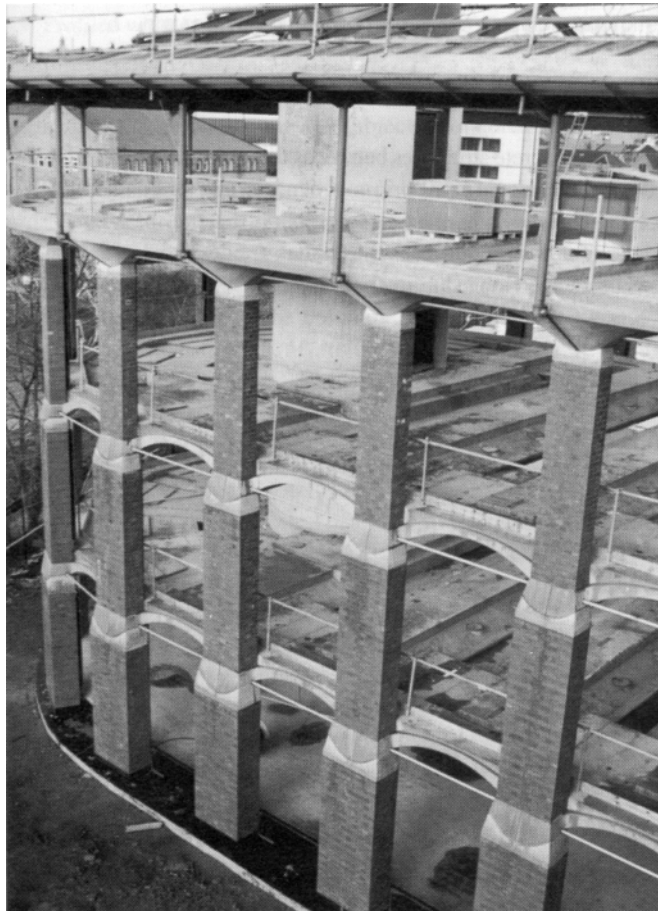

Figure 1.24 Prefabricated brick piers and concrete floor units.

Figure 1.25   Profiled white concrete columns supporting a steel roof (courtesy of Crendon Structures Ltd).

The components were manufactured as single long-span units, with dummy joints to give the appearance of numerous individual arched units in the short-span direction. The alternative design would have been to support vaulted panel units on longitudinal beams spanning between the piers. Structural and architectural variety was incorporated into this scheme at several levels. L-shaped blocks and quadrangles were joined with towers of glass blocks, which accommodated stairs and also acted as flue stacks to drive natural ventilation. Energy-saving features included structural components projected beyond the façade to shade windows, and the thermal mass of the floors limited peak summer temperatures.

The advantages of using single-piece visual concrete components are also evident from the photograph in Figure 1.25 of a building where cantilever columns support a lightweight steel roof. Concrete strengths of 50 MPa were achieved using white Portland cement and 10 mm down limestone aggregates. The structural capacity is no different from a column made from ordinary grey concrete.

1.5   Precast Superstructure Simply Explained

1.5.1   Differences in precast and cast-*in situ* concrete structures

Cast-*in situ* concrete structures behave as three-dimensional (3-D) frameworks. Continuity of displacements and equilibrium of bending and torsional moments, and shear and axial forces, are achieved by reinforcing the joints so that they have strengths equal to the members. However, for design purposes the frames are usually designed in 2-D plane stress, although the presence of the floor slab in the third dimension will affect the manner in which the plane frame behaves. This is particularly important in the end bay, where the beams and columns in the 2-D idealisation may be subject to torsional stresses and biaxial bending, respectively.

Figure 1.26(a) shows the approximate deflected shapes and bending moment distributions for a two-storey continuous frame subjected to gravity and horizontal loads. The foundations are assumed to be fully encastred in this example, which we may call frame F1. The relative magnitudes of the moments in the beams and columns depend on the relative stiffness (EI/L) of the columns and beams meeting at a joint. The joints depend on having equal strength M_1 to the members.

Rigid joint moment M₁

Member moment capacity M₁

(a)

Large sway and second order bending moments

No moment to beam
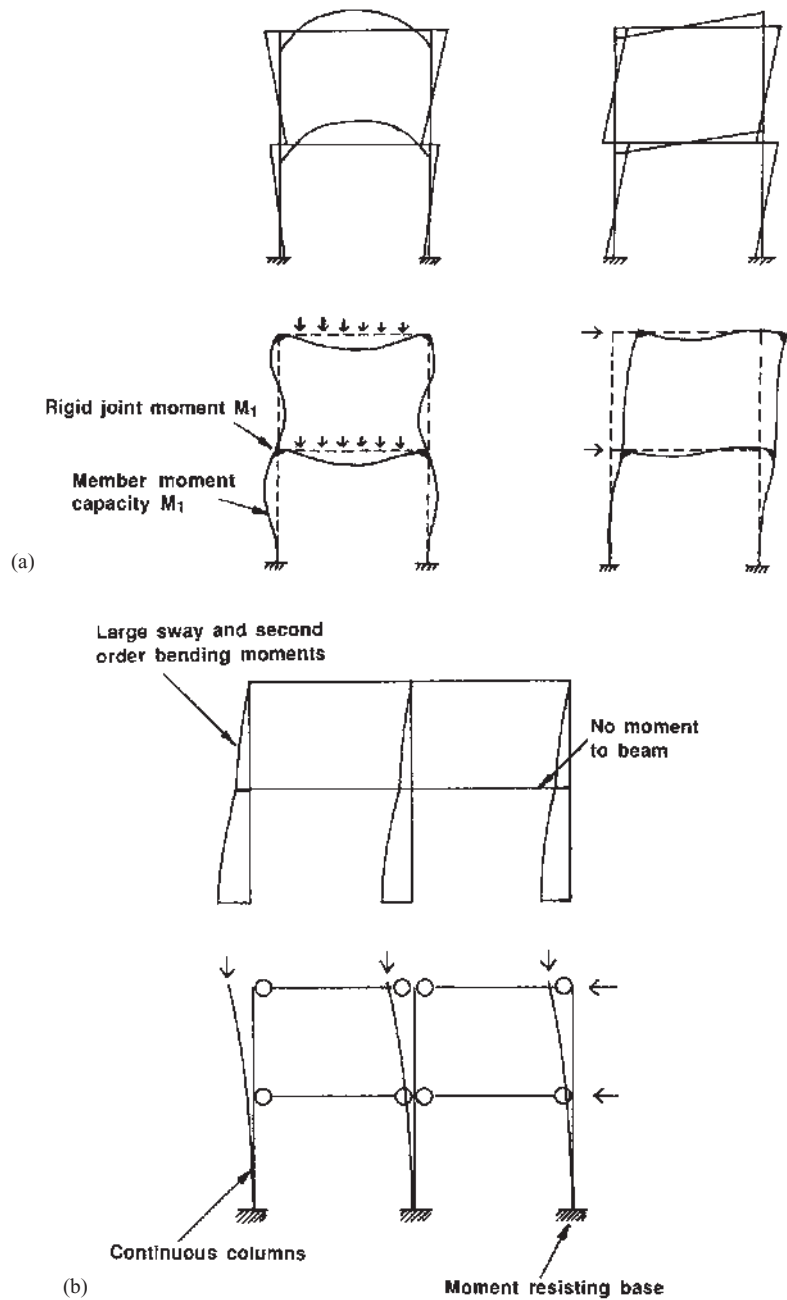
Continuous columns

Moment resisting base

(b)

Figure 1.26 Moments and deflections: (a) in continuous frameworks and (b) in a unbraced structure.

If the strength of the joints in either of the beams in this frame were deliberately weakened to M_2 (e.g. by omitting reinforcement), the behaviour of the frame (called F2) would be equal to that of frame F1 up to the point where the moment in the joint reached M_2. Upon further loading the frame F2 would therefore develop plastic hinges at the joints. The difference between the bending moments in the column above and below the joint would be equal to the moment in the beam, M_2.

Taking this to the limit, if the joints were weakened to $M_3 = 0$, a pinned connection would result and the deflections and moments would be as shown in Figure 1.26(b). Note how the column moments have increased to allow for the fact that the moments can no longer be distributed into the beams. The difference between the bending moments in the upper and lower column at the joint is now zero. This is how a 'skeletal' precast structure behaves, because the beam–column connections are 'pinned' and the structure can be readily precast, with the individual components being bolted, dowelled or welded together on site. If there are no other stabilising elements in the structure, the ends of the columns at the foundations must be encastred.

The key elements in the design of skeletal structures are given by Elliott in journal papers and textbooks [1.23–1.25].

The term 'pin-jointed' refers essentially to the manner in which connections between columns, beams and floor slabs are made. The form of construction does not lend itself to seismic design and, conversely, in seismic zones fully rigid frame connections do not lend themselves to the safest and most economical use of precast concrete. Between these two extremes are semi-rigid connections, and design methods are now available for the post-elastic regime to be considered for the ultimate limit state design of columns in precast structures [1.26].

In buildings of more than about three storeys the horizontal sway deflections may become excessive, so that additional bracing must be used. Thus stability walls, cores or other forms of bracing are used. The usual practice is to place the stabilising units around lift shafts or stairwells so that the open-plan office space is not interrupted. The thickness of the walls varies from 125 mm to about 250 mm, depending on the size of the building. It is possible to cast door or window openings in walls, provided that viable force paths are maintained.

The structure is now classed as 'braced'. Figure 1.27 and the foundations may now be pinned. This simplifies foundation construction considerably and means that braced precast structures can be erected over *in situ* concrete retaining walls, beams, etc. and poor ground. The stabilising elements are so massive that the stiffness of the frame elements and connections is not important. Bending moments due to sway are small and columns can only deflect between floors as pin-ended struts.

In all cases, braced or not, horizontal wind loads are transmitted to the vertical frames through the precast floor, sometimes using unscreeded slabs, as though the floor were a deep beam on plan. One-way spanning, prestressed (or reinforced) precast floor slabs are seated on, or are recessed into, beams.
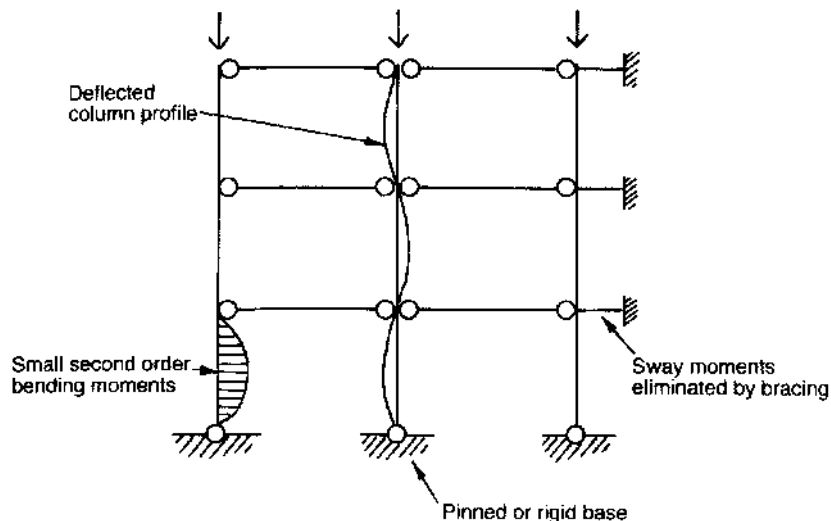

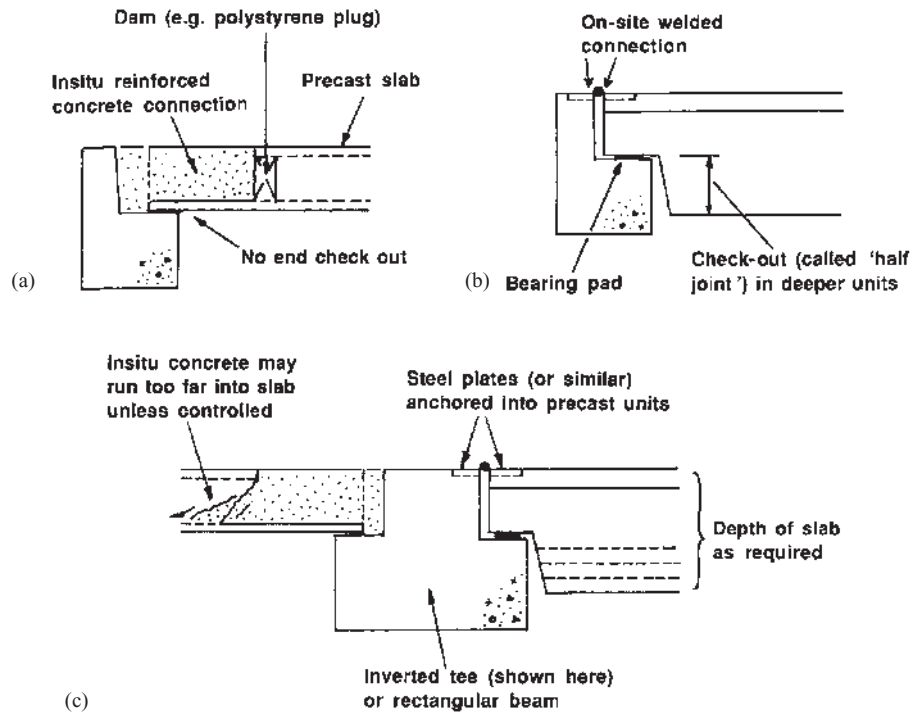**Figure 1.27**   Moments and deflections in a braced structure.

Dam (e.g. polystyrene plug)

On-site welded connection

Insitu reinforced concrete connection

Precast slab

No end check out

(a)

(b) Bearing pad

Check-out (called 'half joint') in deeper units

Insitu concrete may run too far into slab unless controlled

Steel plates (or similar) anchored into precast units

Depth of slab as required

(c)

Inverted tee (shown here) or rectangular beam

Figure 1.28 Detail of slab to beam bearing: (a) edge beam to hollow-core slab; (b) edge beam to double-tee slab; (c) internal beam to slab connections.

The structural zone is sometimes less than in an equivalent fire-protected steel frame, often showing a saving of 100 mm per floor [1.27, 1.28]. The slab bearing shown in Figures 1.28(a), 1.28(b) and 1.28(c) is designed as a simple pinned joint, despite the presence of the reinforced concrete *in situ* infill which penetrates into the floor slab and obviously provides some moment restraint. Beam design is to ordinary reinforced (rc) or prestressed concrete (psc) principles, and composite construction with the floor slab is occasionally appropriate. Precast staircases and landings are designed as inclined solid rc or psc units, and are omitted from the floor plate action.

Beams are connected to columns and walls using connectors that are, in the main, designed as pinned joints: Figure 1.29(a). Eccentric loading is applied to the column, which is continuous at the connection, and the bending moment is distributed in the column according to simplified 2-D frame analysis. Alternatively the beams may be connected on the tops of columns, Figure 1.29(b), or made continuous across the top of single-storey columns, Figure 1.29(c), and connected to the neighbouring beam away from the highly stressed column connection. Columns are considered continuous at floor joints, even though mechanical connections, called 'splices', are often made at this level, as shown in Figure 1.30.

In this book, architectural aspects will be dealt with in Chapter 3, the frame analysis and design methods in Chapter 4, component design in Chapters 5 and 6, the design of connections in Chapter 7, and the analysis and design for horizontal stability will be in Chapter 8.

1.5.2 Structural stability

Structural stability is the most crucial issue in precast concrete design, because it involves the design both of the precast concrete components and of the connections between them.
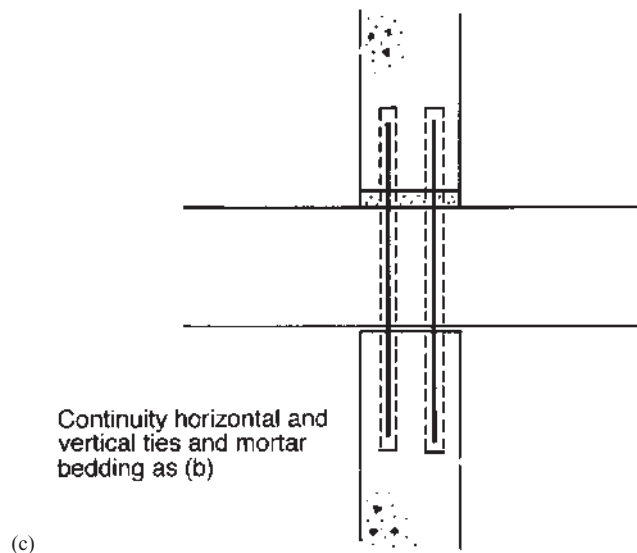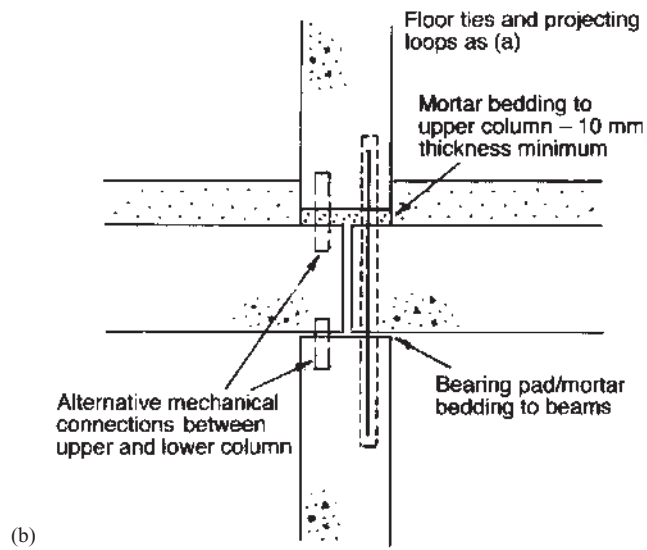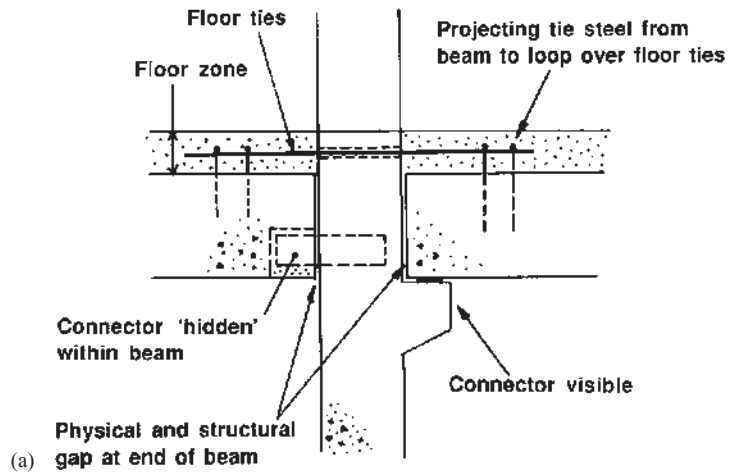
Figure 1.29 Beam to column connections: (a) beam to column connection – continuous column; (b) beam to column connection – discontinuous column and beam; (c) beam to column connection – continuous beam.
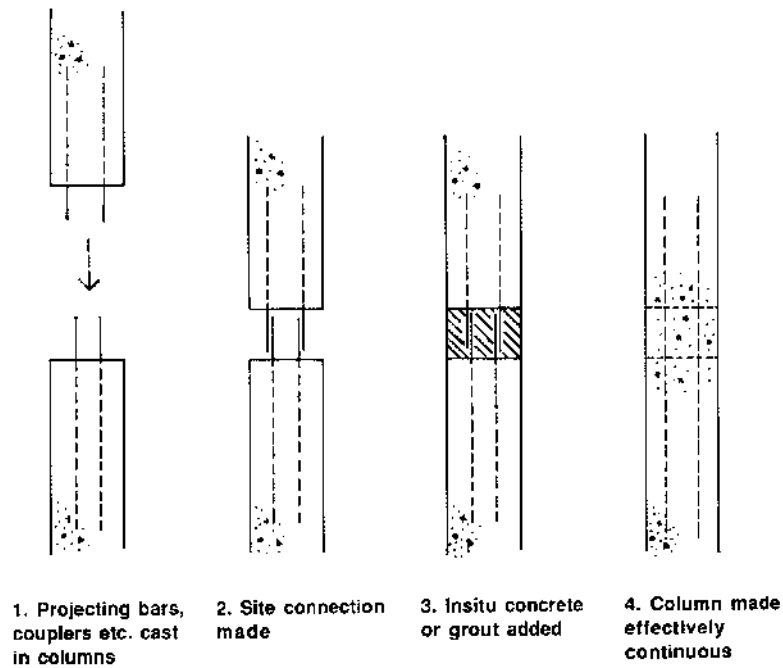
1. Projecting bars, couplers etc. cast in columns   2. Site connection made   3. Insitu concrete or grout added   4. Column made effectively continuous

Figure 1.30 Principle of column splices. The substructure member could also be a wall or beam.

The first design features are stability and robustness. Precast systems are scrutinised by checking authorities more for structural stability, integrity, resistance against abnormal loading and robustness than for the design of individual precast components (slabs, beams, etc.), which usually have adequate factors of safety. In general 'stability' means adequate resistance against side-sway, and 'integrity and robustness' means correct joint design, attention to details and the prevention of progressive collapse. Chapter 9 describes in detail how to achieve structural integrity and resistance to accidental loading.

The problem is to ensure adequate ultimate strength and stiffness, but more importantly to ensure that the failure mode is ductile. Large factors of safety are less relevant to the overall performance criteria if brittle, or sudden, failures result. Brittle failures involve the rapid release of large amounts of energy.

Two design stages are considered:

- **Temporary stability during frame erection:** This has certain implications for design, e.g. the axial load capacity of temporary column splices must be greater than (at least) the self-weight of the upper column before the *in situ* infill grout has hardened and rendered the splice permanent. Chapter 10 will deal further with this topic.
- **Permanent stability:** This may be subdivided into four further stages:
  - horizontal diaphragm action in the precast floor slab;
  - transfer of horizontal loading from the floor slab and into the vertical bracing elements and thus into the foundations;
  - component design; and
  - connection and joint design.

The contribution to the lateral strength and stiffness of the structure from the *in situ* reinforced concrete infill strips is paramount. These strips provide the necessary tie forces that eliminate relative displacements between the various parts of the frame and ensure interaction between the components. The general idea is shown in Figure 1.31. These positions are not always easy to define, and no
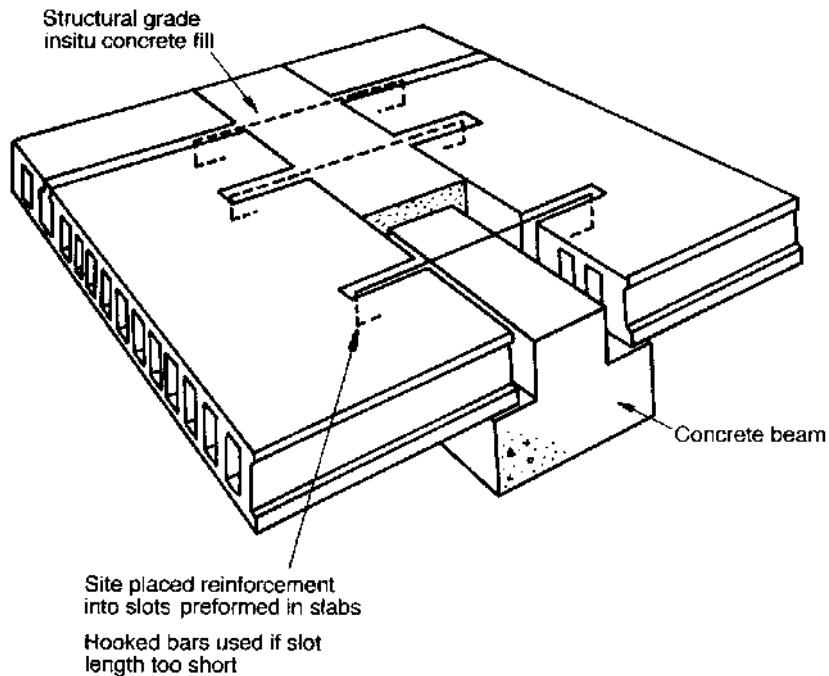
Figure 1.31 Continuity ties in the *in situ* concrete infill.

experimental work has addressed this problem directly. Engineers are cautious not to allow service openings or novel connections in these highly sensitive areas.

Stability may be achieved in several ways, but in the vast majority of cases it is based on either:

(1) an 'unbraced' (or 'sway') structure, Figure 1.26b, where stability is provided within the skeletal structure by cantilever action of the columns; or
(2) a 'braced' structure, Figure 1.27, where resistance against horizontal loading is not provided by the skeleton of the beams, columns and slabs. In other terminology, e.g. using *in situ* concrete, this would be called a 'no-sway' frame.

Combinations of the above are possible, giving what is then known as a 'partially braced' structure: Figure 1.32. It is perfectly reasonable for the frame to be braced in one plane and rely on column action in the other plane, particularly if the building is long and narrow. The design of these structural systems will be dealt with in Chapter 8, but it is important that sufficient bracing elements, e.g. walls, service shafts, columns, etc., are provided at the very outset of design, not at the end.

The positioning of shear walls is often a contentious issue. A conflict with architectural requirements as to the number of walls and their positions is usually the biggest problem. In general, it is necessary to 'balance' the flexural resistances (i.e. summation of stiffness) of the walls in the structure (in two mutually perpendicular directions) to avoid torsional effects as far as is reasonably practical. A design method for the different types of wall used in precast concrete frames is given in Chapter 8.

1.5.3 Floor plate action

Floor plate action means the transfer of horizontal loading across a building. It is sometimes called 'diaphragm action', because the floor is a relatively thin membrane. There has been a considerable
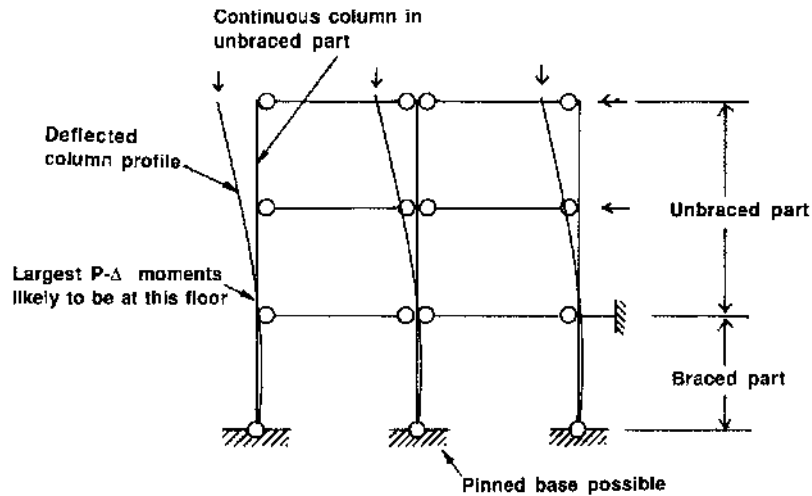
Figure 1.32 Partially braced structure.

debate about whether a precast concrete floor, consisting of a large number of discrete elements, tied together only around their edges as shown in Figure 1.33(a), can in fact be used as a floor diaphragm. The ability of discrete components to act compositely as a single diaphragm has now been demonstrated [1.29, 1.30]. Various structural models have been proposed for the shear transfer mechanism, which relies on the development of clamping forces generated in the tie bars placed in the *in situ* concrete strips at the ends of the slabs. Shear forces parallel with the span of the flooring units are transmitted to shear cores or other stabilising features by deep beam action. Shear forces perpendicular to the span of the floor slab are carried to successive bays of flooring by the shear friction reinforcements in the *in situ* concrete strips in the beam to slab connections.

If this cannot be achieved, for example in precast floor units with discontinuous or thin flanges, then a structural topping screed is used, as shown in Figure 1.33(b).

1.5.4 Connections and joints

The term 'connection' refers to major structural connections between precast components, whereas the term 'joint' is used to describe a more simplified jointing between components. For example, the entire junction between beams and column is called a connection, whereas the halving detail between landing and stair flight is classed as a joint.

Connections between components are the most important factors influencing the design, construction and in-service behaviour of precast structures. The many different types of connection used by designers make it difficult to generalise on rules and guidance notes, particularly because engineers practise different methods of making the same type of connection. The major connections are between:

- column or wall to the foundation
- column to column
- beam to column or wall
- beam to beam
- slab and/or staircase to beam or wall
- slab to slab
- structural steelwork, *in situ* concrete, timber and masonry to precast concrete components.
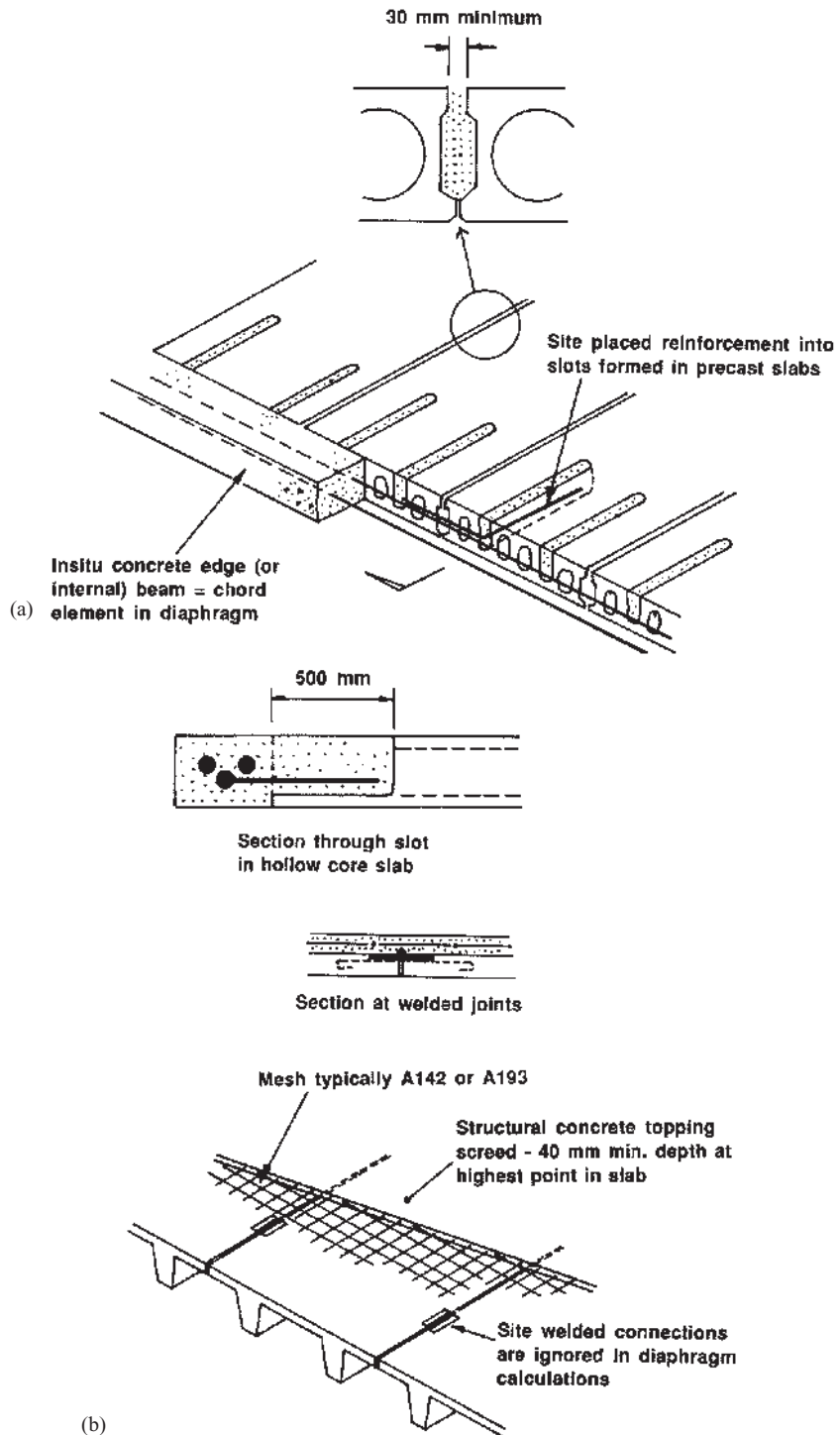
30 mm minimum

Site placed reinforcement into
slots formed in precast slabs

Insitu concrete edge (or
internal) beam = chord
(a) element in diaphragm

500 mm

Section through slot
in hollow core slab

Section at welded joints

Mesh typically A142 or A193

Structural concrete topping
screed - 40 mm min. depth at
highest point in slab

Site welded connections
are ignored in diaphragm
calculations

(b)

Figure 1.33   Basic principles of the precast floor diaphragm. (a) Precast concrete floors only and (b) precast floors with and *in situ* structural topping.

Joints are required mainly (but not exclusively) to transmit compression, tension and/or shear stresses. Bending and torsion moments can usually be resolved into these three components. Typical forms of joint construction involve wet grouting, dry-pack grout, adhesives, neoprene pads, welds, bolted cleats, dowels and screwed rods. Examples of joints are:

- bearings between slabs and beams or walls
- interfaces between precast and *in situ* concrete, e.g. structural topping
- scarf or halving joints between stair components
- trimmer angles forming holes in floor decks
- site erection aids (temporary or permanent).

1.5.5   Foundations

Loads and moments in columns are transferred to *in situ* concrete foundations through deep pockets, base plates or grouted sleeves. Wall loads and moments are usually spread over such a large area that the connections are made up of simple compression and tension joints.

The various options for foundations are shown in Figures 1.34(a), 1.34(b) and 1.34(c). The available depth of foundation may be the deciding factor in the type of stabilising system used in the structure, e.g. a shallow footing requires that a braced structure is used. The main criteria for foundations are simplicity in design and ease of erection of the precast superstructure.

Foundation design is much simplified in braced frames where only vertical axial loading is present, with small bending moments due to the carry-over couples resulting from eccentric loads at the first-floor beam connection. Simple pad footings are used in most instances, although the columns in a braced frame may be spliced onto the tops of *in situ* concrete retaining walls.

Moment-resisting foundations are required in unbraced frames. These may be very expensive in poor ground, and because of this it is often advisable to liaise with other bodies on the overall economics of unbraced structures. Structures exceeding three storeys or about 12 m in height should certainly be designed as braced. Precast box foundations have been tried, but the preparation of level ground for them outweighs most of the advantages.

1.6   Precast Design Concepts

1.6.1   Devising a precast solution

In their recent book, Bruggeling and Huyghe [1.31] state: *Prefabrication does not mean to 'cut' an already designed concrete structure into manageable pieces* . . .

The correct philosophy behind the design of precast concrete multi-storey structures is to consider the frame as a total entity, not an arbitrary set of elements each connected in a way that ensures interaction between no more than the two elements being joined. Thus it is clear that all the aspects of component design and structural stability are dealt with simultaneously in the designer's mind. The main aspects at the preliminary stage include:

- structural form
- frame stability and robustness
- component selection
- connection design.

These items cannot be dealt with in isolation. For example, the nature of the column–beam connection dictates the arrangement and function of the reinforcement in the ends of beams, and the manner in which floor slabs are connected to edge beams influences the torsional behaviour of the beam. Frame design is therefore an integrated process in which many of the iterative steps are not
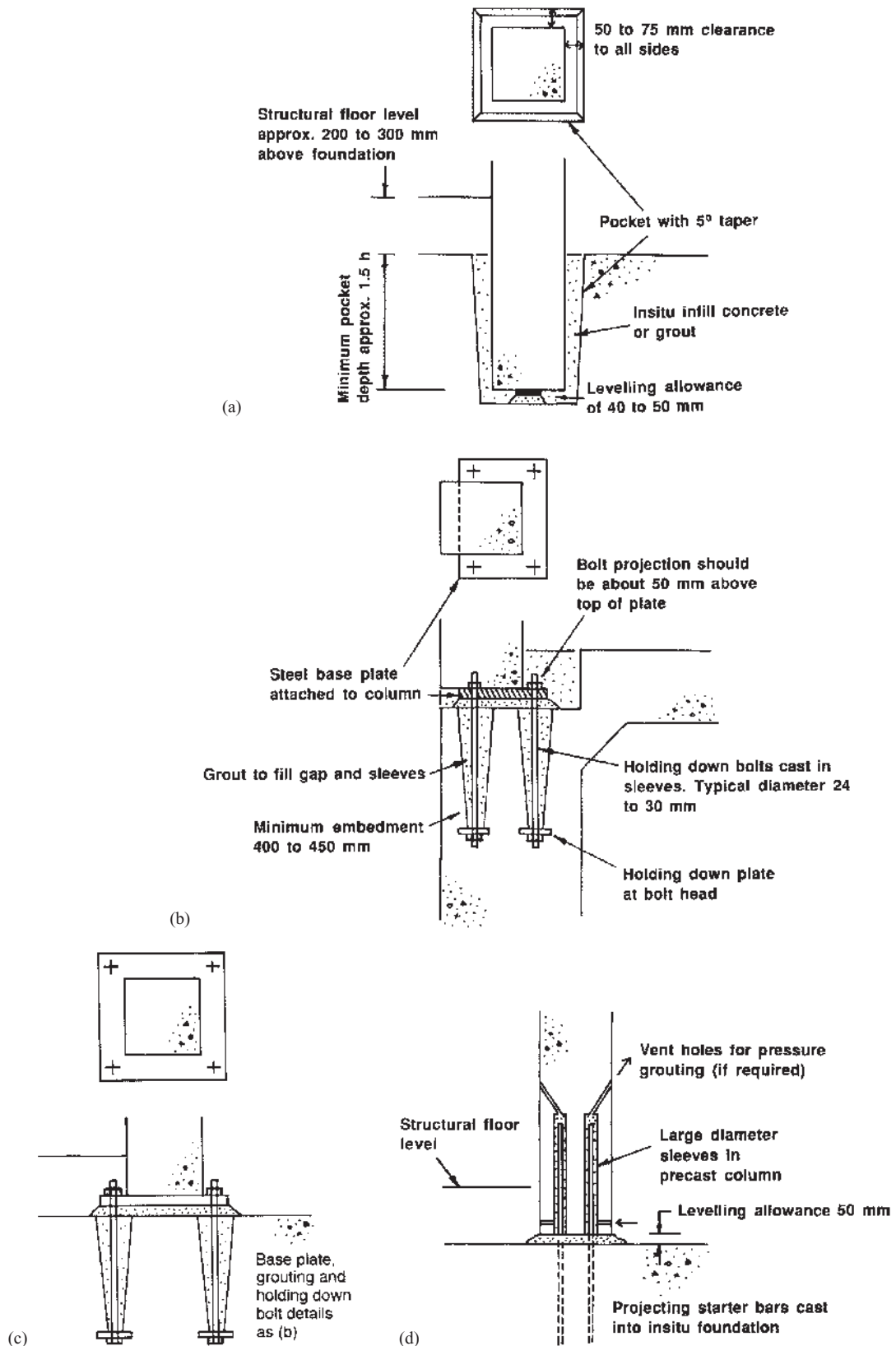
Figure 1.34 Alternative methods of making a column – foundation connection. (a) Pocket in pad footing; (b) base plate to *in situ* wall. Alternative methods of making a column-connection. (c) Base plate to pad footing. (d) Grouted projecting bars in pad footing or basement wall.

so obvious, because they are now hidden within the natural evolution of the design, detailing and site erection procedures. It must also be remembered that two additional procedures, namely manufacture and site erection, are also directly influential in making design decisions.

The correct order for dealing with the framing plan is:

- positions of construction joints
- positions of lift shafts and stairwells, which provide the locations for shear walls and/or cores
- positions of shear walls (if more than three storeys, or if on poor ground)
- positions of columns
- structural floor zones
- cantilevers
- direction of span and length of span of beams and slabs at all floors, and roof if different in plan
- use of non-preferred structural components
- use of structural precast concrete cladding
- positions of service areas and major holes in floor slabs
- staircases and precast lift-shaft boxes.

The main building function is assessed with respect to:

- architectural and aesthetic requirements
- loadings, i.e. dead, imposed dead, live, wind, seismic (occasionally), lateral load due to temperature, creep, shrinkage, and miscellaneous loads such as vibrating or mobile machinery
- fire and durability
- building regulations and codes of practice
- design guides and manuals.

The first task is therefore to establish an economical plan layout for the optimisation of the minimum number of the least-cost components versus overall building requirements. The optimum is usually found in a rectangular grid where the beams span in a direction parallel with the greater dimension of the frame. As a general rule, the minimum construction depth, for the same span and loading, is achieved when the ratio of the floor span/beam span = 1.5 to 2.0.

Figure 1.35 shows three options, for each of which there is a weighted function reflecting design, detailing, manufacture and construction effort and relative costs. There is a delicate balance between the architect's wishes and the precaster's pragmatism. Precast component selection will follow logically and simply once the optimum layout has been achieved. So too will service routes, as the most favourable option shown above will allow easy highways for pipes and cables.

The optimum solution in terms of building efficiency and cost is not always obvious. Consider the three different floor layouts shown in Figure 1.35. Scheme A (Figure 1.35(a)) may appear to give the best solution in terms of number of building components, and this is certainly the most favoured solution. However, as the data in Figure 1.36 show, scheme C (Figure 1.35(c)) offers the lowest ratio of floor zone to bay area, with a saving of about 100 to 150 mm per floor. If, in this case, the cost of the peripheral cladding is such that the saving in surface area exceeds the additional expense of the precast components, scheme C would be the cheapest. This simple example must be treated with caution, as there are many more variables affecting the final cost.

The rules for determining whether a structure is to be braced or unbraced are fairly clear-cut, and they affect both prefabricated steel and concrete frames alike. Table 1.3 gives guidance.

The robustness of the structure must also be considered at the conceptual stage, particularly if a structural floor topping is not being used. Making sure that all the necessary peripheral and internal ties can indeed be placed and be continuous and fully anchored at their ends is a design exercise in its own right. Dangerous congestion of ties can be avoided if the layout of beams is thought through at this early stage.
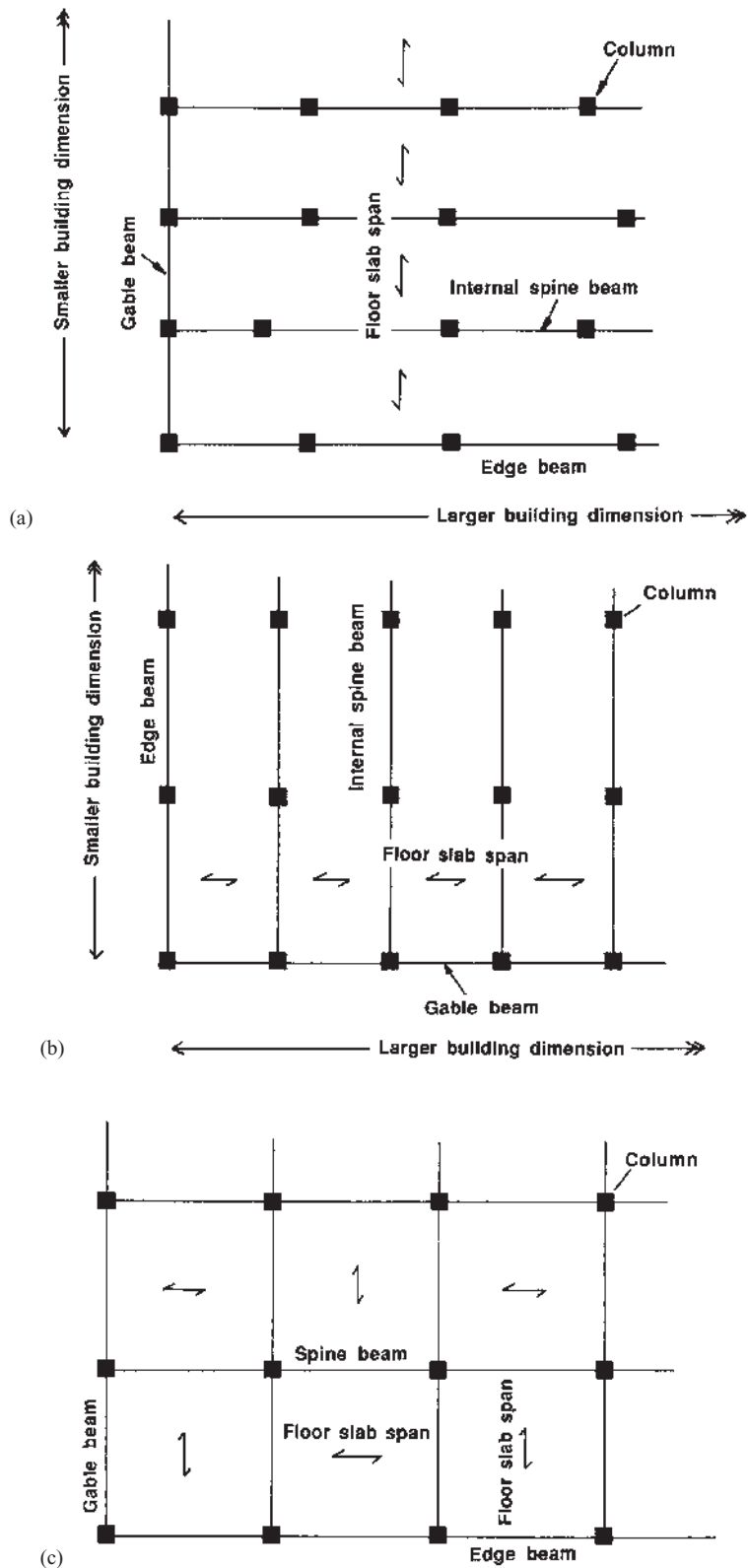
Figure 1.35 Options for layout of primary beams and floor slabs. (a) Scheme A: one-way spanning system with beams spanning in the larger building dimension. (b) Scheme B: one-way spanning system with beams spanning in the smaller building dimension. (c) Scheme C: two-way spanning system.
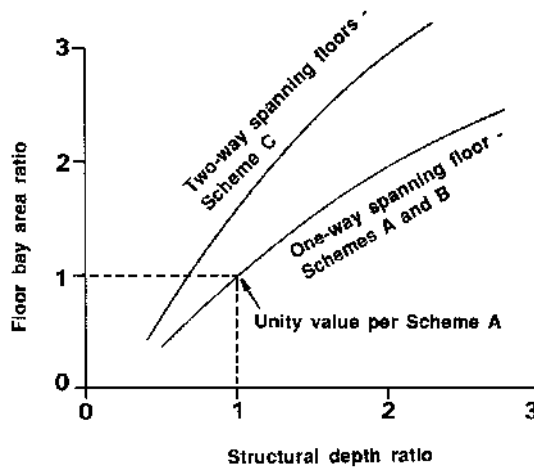
Figure 1.36   Structural zones versus bay area for the structural layouts shown in Figure 1.35.

Table 1.3   Storey heights of precast concrete frames and type of bracing [1.32]

Approximate economical range for number of storeys	Type of frame	Bracing element(s)
2		Cantilevered columns
Up to 3	Unbraced	Cantilevered columns (roof load small)
3 or 4 to 6		Precast wind posts (deep columns)
Up to 4		Steel cross bracing
Up to 5 or 6		Brick or block infill walls
3 to 5 or 6		Precast hollow-core infill walls
3 to 10	Braced	Precast solid infill walls
3 to 10		Precast solid cantilever walls
3 to 12		Precast hollow-core shear walls
10 to 15		Precast concrete shear box(es)
15 to 20		*In situ* conrete shear core(s)
Up to 5		Brick or block infill walls
5 to 10	Partially braced*	Precast infill walls
5 to 12		Precast hollow-core shear walls

* Uppermost one or two storeys unbraced.

1.6.2   Construction methods

Precast frames can greatly improve buildability, because many of the sensitive site operations are moved to the protective environment of the factory. Seasonal variations are less critical to site progress, and are totally nullified at the factory. Depending on the circumstances of the design, size and complexity of the building and the conditions at the construction site (i.e. access), prefabrication has the following approximate savings over cast-*in situ* construction at the site:

* scaffolding material and labour to erect scaffold   80–90%
* shuttering and formwork   90–95%
* delivery and pouring wet concrete*   75–95%

* Lower value where structural floor toppings are used.

- delivery and fixing of loose reinforcement   90–95%
- time of construction of superstructure (above foundations)   25–50%
- total construction time   10–30%
- site labour on superstructure   75–90%
- total site labour   50–75%

One of the key issues is programming the deliveries to site so that the construction team is not under pressure to construct hastily, nor to fix any of the components out of sequence. This could impair the temporary stability of the structure, as the height to the centre of the mass of the concrete above the level at which the frame is stable could be prohibitive to further progress. The rule is that components should not be fixed more than two storeys ahead of the last floor to be fully tied into the stabilising system. This allows time for the *in situ* concrete at the lower levels to mature. Theoretically it is possible to erect precast frames of up to about seven or eight storeys in height before the components are permanently structurally tied together; however, there is evidence that this is folly, and it leaves no room for error.

On-site construction methods have a significant influence on design. Most of these concern connection details, jointing materials, and temporary stability. In many instances the construction sequence will dictate the design of the frame. Often, the positions of shear walls, sizes of beams, spans, etc. can be finalised only when the construction programme is finalised, and the type and capacity of the crane is agreed. Significant economies can be achieved if the designer takes into account all the benefits available on site. In this respect construction sequences are self-defined, with columns, walls, beams, slabs and staircases being the obvious progression. However, the main decision to be made at the design stage is a logistical one.

The two main options are:

- completion of frame floor by floor, Figure 1.37
- completion of frame to roof block by block, Figure 1.38.

Figure 1.37   Construction sequence floor by floor (courtesy of Trent Concrete Ltd).

Figure 1.38   Construction sequence block by block (courtesy of Blatcon Ltd).

The following points should be considered:

- positions of shear walls and maturity of connections, which may dictate site progress
- possibility of temporary instability and the need to design some of the key components to eliminate it unless foundation fixity is available
- availability and/or positioning of equipment to transport and erect every component
- size and weight of components
- safety and speed of construction
- tolerances for economical construction.

The vast majority of precast structures are erected by fixing gangs with many years of experience in handling precast concrete. Many precasting companies employ their own fixing teams, and that is obviously beneficial as far as feedback to the design office and factory is concerned. Information about adequate tolerances is essential to the smooth running of a site. The UK Precast Concrete Industry Training Association [1.33], the UK Structural Products Association [1.34] and the UK Precast Flooring Federation [1.35] have published guides to the safe erection of precast concrete frames, cladding and flooring.

Programming can increase the overall speed of construction by allowing parts of the building to be released to following trades while work continues on erecting the rest of the precast structure. To construct bays to the full height while backing out of the building enables the structure to be released bay by bay. The alternative method, to construct floor by floor, releases lower floors while work continues on the erection of the floors above. Access can often be gained within two or three weeks of starting precast construction.

Table 1.4   Site fixing rates for generalised frame layouts

Frame layout	Example bay area* (m²)	Site fixing rate per week (m²)
Rectangular grid	100	1000–1500
Rectangular grid	40–50	600–800
Non-rectangular grid	100	900–1200
Non-rectangular grid with staggered columns	40–50	300–400

* A 'bay' is defined by the area enclosed between four columns.

The construction cost element, which includes transportation costs, varies with building size, number of storeys and the structural grid. In general it is about 8 to 12 per cent of the cost of the precast structure (not the finished building). The information may sometimes be deceptive, as the cheapest site cost may have hidden extras, e.g. the cheapest beams are the longer, which means greater depth and hence a greater structural zone.

Speed of construction is a major consideration in most building projects and it is there that the design of precast structures should be carefully considered. This advantage is maximised 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 also be used in complex and irregular structures, although it may not then provide the same efficiency of construction as a rationalised design. The fixing rates shown in Table 1.4 vary depending on the shape and size of the structure, i.e. the number of linear components such as beams and columns per unit area of the building. The data refer to typical site progress using a fixing gang of no more than five persons, and one crane. A precast solution can stretch to a considerable number of non-standard details, say about 30 to 40 per cent of the total, before the fixing rates reduce dramatically.

The rates given in Table 1.4 are presented in terms of the floor area covered within the beam and column grid: for example, 100 m² could be a 12.0 m span floor carried on 8.5 m span beams, and so on. They do not include major volumes of cast-*in situ* concrete, such as structural toppings. The rates are affected by vertical transportation time, typically 3–5 minutes extra per item for buildings of more than about 6–10 storeys, respectively. The data also reflect the different environmental factors, e.g. winter versus summer weather and daylight hours, regionally available plant, particularly 5–10 tonne lifting capacity tower cranes, and ground conditions from green field to congested city sites. The data were collected from UK companies between 1990 and 1996.

The three-storey building shown in Figure 1.39 [1.36] achieved the following fixing rates on a 6 m × 6 m grid layout:

- columns (14.5 m long)   30 per week
- beams (6 m long generally)   45 per week
- floor slabs (6 to 17 m span)   2335 m² per week
- façade panels   750 m² per week

The structural form of a precast structure may be influenced by the manner in which it is built, the site access for long vehicles, and the capacity of the craneage. One of the most important decisions pertaining to temporary stability is the availability of a stiff shear core, or shear walls (or other forms of bracing) in two orthogonal directions at all times during construction. This may not seem to be a problem, but consider this structure shown in Figure 1.40, which would be constructed to its full height, bay by bay. If the shear core were available only up to point X, the previous bays would be unstable. In low-rise buildings of fewer than four storeys the columns can be stabilised by propping

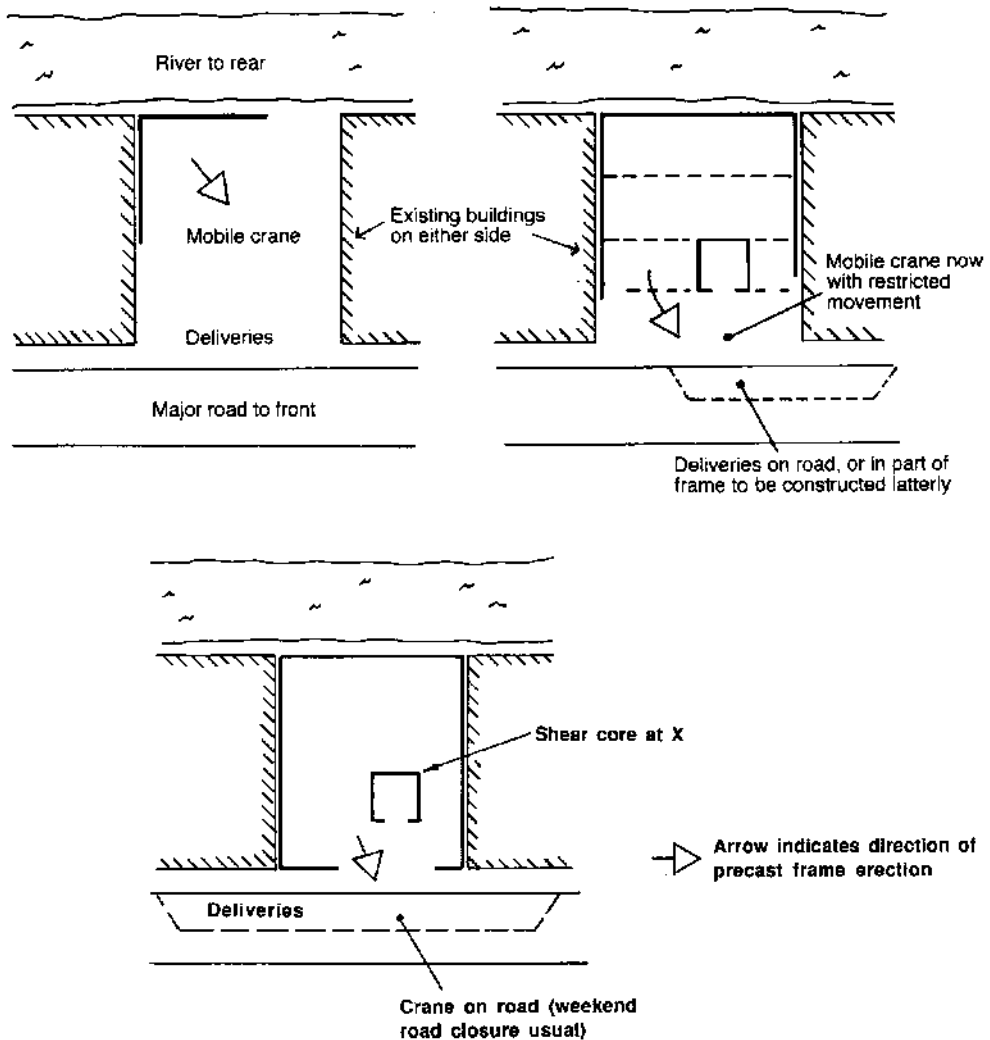**Figure 1.39** Skeletal frame (courtesy of B + FT Wisenbaden).

Figure 1.40 Preferred sequence of erection on confined sites.

at the ground level (assuming a sound footing is available, which is not always the case). However once a column splice is included, permanent stability is required.

In all cases the precast frame is correctly aligned and levelled as each floor is completed. Following trades may take up residence at each floor level immediately after the floor level above has been completed. Precast frames may lend themselves to partial completion and a phased hand-over. This depends very much on the form of the structure, and the divisions are dictated by the positions of fully stabilised subframes.

The construction programme may be dictated by the sequence in which components can be fixed. If the plan area is small, say less than 300 m², it is possible that the fixing rate can progress faster than the maturity rate of the connections. It may then be advantageous to design a connection that has no restriction on maturity, e.g. a bolted or welded plate, rather than specifying one with a cast-*in situ* 'stitch'.