18 Concrete Building Systems

18.1 Introduction

Most of the material in the preceding chapters has pertained to the design of reinforced concrete *structural elements*, e.g., slabs, columns, beams, and footings. These elements are combined in various ways to create *structural systems* for buildings and other construction. An important part of the total responsibility of the structural engineer is to select, from many alternatives, the best structural system for the given conditions. The wise choice of structural system is far more important, in its effect on overall economy and serviceability, than refinements in proportioning the individual members. Close cooperation with the architect in the early stages of a project is essential in developing a structure that not only meets functional and esthetic requirements but exploits to the fullest the special advantages of reinforced concrete, which include the following:

Versatility of form. Usually placed in the structure in the fluid state, the material is readily adaptable to a wide variety of architectural and functional requirements.

Durability. With proper concrete protection of the steel reinforcement, the structure will have long life, even under highly adverse climatic or environmental conditions.

Fire resistance. With proper protection for the reinforcement, a reinforced concrete structure provides the maximum in fire protection.

Speed of construction. In terms of the entire period, from the date of approval of the contract drawings to the date of completion, a concrete building can often be completed in less time than a steel structure. Although the field erection of a steel building is more rapid, this phase must necessarily be preceded by prefabrication of all parts in the shop.

Cost. In many cases the first cost of a concrete structure is less than that of a comparable steel structure. In almost every case, maintenance costs are less.

Availability of labor and material. It is always possible to make use of local sources of labor, and in many inaccessible areas, a nearby source of good aggregate can be found, so that only the cement and reinforcement need to be brought in from a remote source.

Two record-setting examples of good building design in concrete are shown in Figs. 18.1 and 18.2.

FIGURE 18.1
View of 311 South Wacker Drive under construction. When completed it became the world's tallest concrete building, with total height 946 ft. (*Courtesy of Portland Cement Association.*)

18.2 Floor and Roof Systems

The types of concrete floor and roof systems are so numerous as to defy concise classification. In steel construction, the designer usually is limited to using structural shapes that have been standardized in form and size by the relatively few producers in the field. In reinforced concrete, on the other hand, the engineer has almost complete control over the form of the structural parts of a building. In addition, many small producers of reinforced concrete structural elements and accessories can compete profitably in this field, since plant and equipment requirements are not excessive. This has resulted in the development of a wide variety of concrete systems. Only the more common types can be mentioned in this text.

In general, the commonly used reinforced concrete floor and roof systems can be classified as one-way systems, in which the main reinforcement in each structural element runs in one direction only, and two-way systems, in which the main reinforcement in at least one of the structural elements runs in perpendicular directions. Systems of each type can be identified in the following list:

(a) One-way slab supported by monolithic concrete beams
(b) One-way slab supported by steel beams (shear connectors are used for composite action in the direction of the beam span)

FIGURE 18.2
MLC Centre in Sydney, Australia, with total height 808 ft.

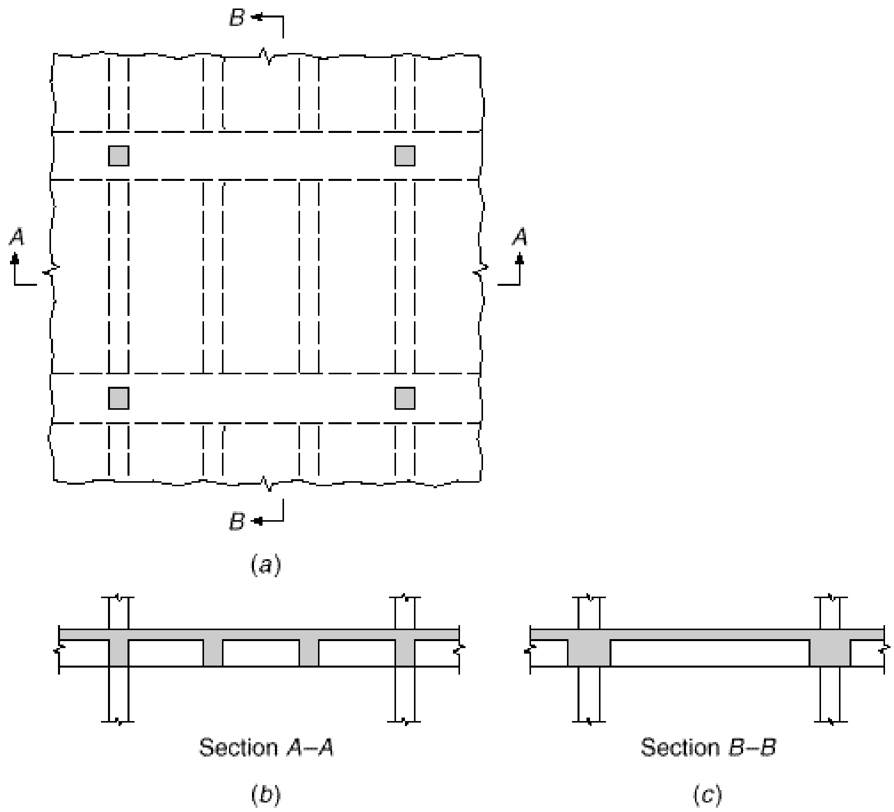
(c) One-way slab with cold-formed steel decking as form and reinforcement
(d) One-way joist floor (also known as ribbed slab)
(e) Two-way slab supported by edge beams for each panel
(f) Flat slabs, with column capitals or drop panels or both, but without beams
(g) Flat plates, without beams and with no drop panels or column capitals
(h) Two-way joist floors, with or without beams on the column lines

Each of these types will be described briefly in the following sections. Additional information will be found in Refs. 18.1 to 18.3. In addition to the cast-in-place floor and roof systems described in this section, a great variety of precast concrete systems has been devised. Some of these will be described in Section 18.5.

a. Monolithic Beam-and-Girder Floors

A beam-and-girder floor consists of a series of parallel beams supported at their extremities by girders, which in turn frame into concrete columns placed at more or less regular intervals over the entire floor area, as shown in Fig. 18.3. This framework is covered by a one-way reinforced concrete slab, the load from which is transmitted first to the beams and then to the girders and columns. The beams are usually spaced

FIGURE 18.3
Framing of beam-and-girder floor: (*a*) plan view; (*b*) section through beams; (*c*) section through girders.



so that they come at the midpoints, at the third points, or at the quarter points of the girders. The arrangement of beams and spacing of columns should be determined by economical and practical considerations. These will be affected by the use to which the building is to be put, the size and shape of the ground area, and the load that must be carried. A comparison of a number of trial designs and estimates should be made if the size of the building warrants, and the most satisfactory arrangement selected. If the spans in one direction are not long, say 16 ft or less, the beams may be omitted altogether, and the slab, spanning in one direction, can be carried directly by girders spanning in the perpendicular direction on the column lines. Since the slabs, beams, and girders are built monolithically, the beams and girders are designed as T beams and advantage is taken of continuity.

Beam-and-girder floors are adapted to any loads and to any spans that might be encountered in ordinary building construction. The normal maximum spread in live load values is from 40 to 400 psf, and the normal range in column spacings is from 16 to 32 ft.

The design and detailing of the joints where beams or girders frame into building columns should be given careful consideration, particularly for designs in which substantial horizontal loads are to be resisted by frame action of the building. In this case, the column region, within the depth of the beams framing into it, is subjected to significant horizontal shears as well as to axial and flexural loads. Special horizontal column ties must be included to avoid uncontrolled diagonal cracking and disintegration of the concrete, particularly if the joint is subjected to load reversals. Specific recommendations for the design of beam-column joints are found in Chapter 11 and Ref. 18.4. Joint design for buildings that resist seismic forces is subject to special ACI Code provisions (see Chapter 20).

In normal beam-and-girder construction, the depth of the beams may be as much as 3 times the web width. Improved economy, however, is achieved by using beams with webs that are generally wider and shallower, coupled with girders that have the same depth as the beams. The resulting girders, more often than not, have webs that are wider than their effective depths. Although the flexural steel in the members is increased because of the reduced effective depth compared with deeper members, the increases in material costs are more than paid for by savings in forming costs (one depth for all members) and easier construction (wider beams are easier to cast than narrow beams). Another key advantage is the reduced construction depth, which permits a reduction in the overall height of the building.

For light loads, a floor system has been developed in which the beams are omitted in one direction, the one-way slab being carried directly by column-line beams that are very broad and shallow, as shown in Fig. 18.4. These beams, supported directly by the columns, become little more than a thickened portion of the slab. This type of construction, in fact, is known as *banded slab construction,* and there are a number of advantages associated with its use, over and above those associated with shallow beam-and-girder construction. In the direction of the slab span, a haunched member is present, in effect, with the maximum effective depth at the location of greatest negative moment, across the support lines. Negative moments are small at the edge of the haunch, but the depth becomes less, and positive slab moments are reduced as well. The increased flexural steel in the beam (slab-band) resulting from the reduced effective depth is often outweighed by savings in the slab steel. Along with reduced construction depth, banded slab construction allows greater flexibility in locating columns, which may be displaced some distance from the centerline of the slab-band without significantly changing the structural action of the floor. Formwork is simplified because of the reduction in the number of framing members. For such systems, special attention should be given to design details at the beam-column joint. Transverse top steel may be required to distribute the column reaction over the width of the slab-band. In addition, punching shear failure is possible; this may be investigated using the same methods presented earlier for flat plates (see Section 13.10).

b. Composite Construction with Steel Beams

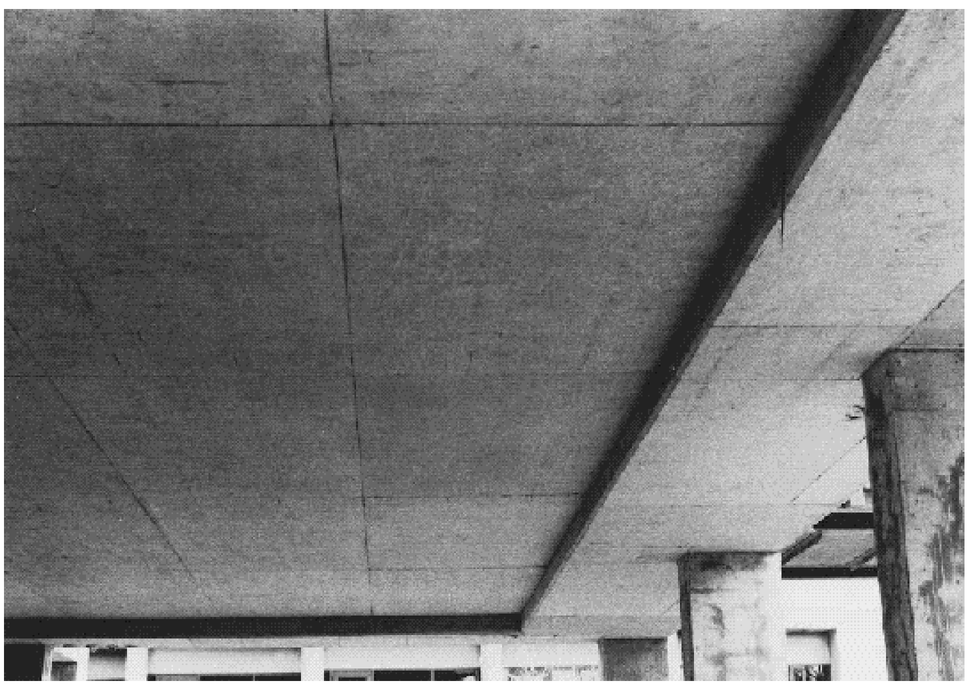
One-way reinforced concrete slabs are also frequently used in buildings for which the columns, beams, and girders consist of structural steel. The slab is normally designed for full continuity over the supporting beams, and the usual methods followed. The spacing of the beams is usually 6 to 8 ft.

To provide composite action, shear connectors are welded to the top of the steel beam and are embedded in the concrete slab, as shown in Fig. 18.5*a*. By preventing longitudinal slip between the slab and steel beam in the direction of the beam axis, the combined member is both stronger and stiffer than if composite action were not developed. Thus, for given loads and deflection limits, smaller and lighter steel beams can be used.

Composite floors may also use encased beams, as shown in Fig. 18.5*b*, offering the advantage of full fireproofing of the steel, but at the cost of more complicated formwork and possible difficulty in placing the concrete around and under the steel member. Such fully encased beams do not require shear connectors as a rule.

FIGURE 18.4
Banded slab floor system.

(·) Interior slab band

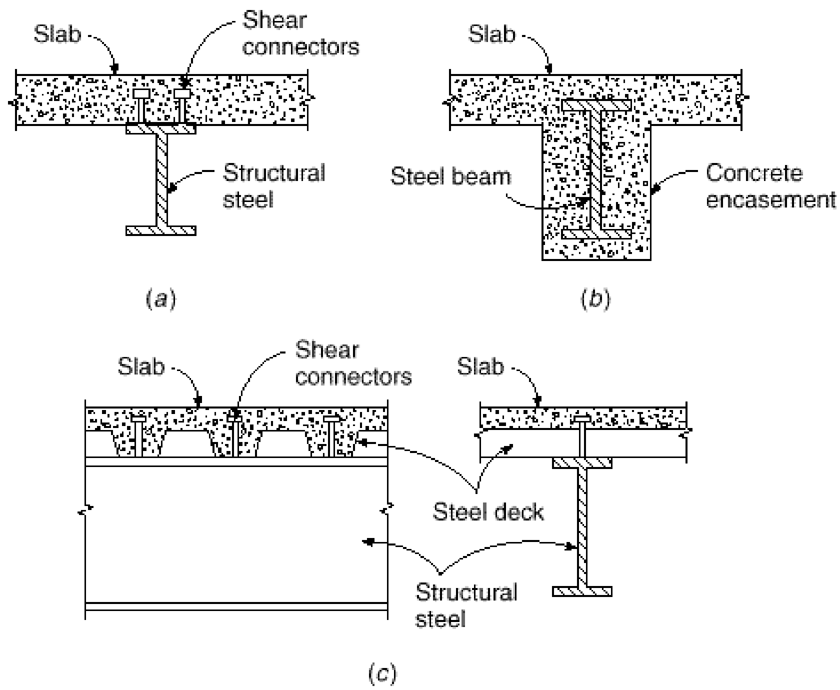


(·) Edge band at exterior column

c. Steel Deck Reinforced Composite Slabs

It is nearly standard practice to use stay-in-place light-gage cold-formed steel deck panels in composite floor construction. As shown in Fig. 18.5*c*, the steel deck serves

FIGURE 18.5
Composite beam-and-slab floor.



as a stay-in-place form and, with suitable detailing, the slab becomes composite with the steel deck, serving as the main tensile flexural steel. Suitable for relatively light floor loading and short spans, composite steel deck reinforced slabs are found in office buildings and apartment buildings, with column-line girders and beams in the perpendicular direction subdividing panels into spans up to about 12 ft. Temporary shoring may be used at the midspan or third point of the panels to avoid excessive stresses and deflections while the concrete is placed, when the steel deck panel alone must carry the load.

d. One-Way Joist Floors

A one-way joist floor consists of a series of small, closely spaced reinforced concrete T beams, framing into monolithically cast concrete girders, which are in turn carried by the building columns. The T beams, called *joists*, are formed by creating void spaces in what otherwise would be a solid slab. Usually these voids are formed using special steel pans, as shown in Fig. 18.6. Concrete is cast between the forms to create ribs, and placed to a depth over the top of the forms so as to create a thin monolithic slab that becomes the T beam flange.

Since the strength of concrete in tension is small and is commonly neglected in design, elimination of much of the tension concrete in a slab by the use of pan forms results in a saving of weight with little change in the structural characteristics of the slab. Ribbed floors are economical for buildings, such as apartment houses, hotels, and hospitals, where the live loads are fairly small and the spans comparatively long. They are not suitable for heavy construction such as in warehouses, printing plants, and heavy manufacturing buildings.

FIGURE 18.6
Steel forms for one-way joist floor.

Standard forms for the void spaces between ribs are either 20 or 30 in. wide, and 8, 10, 12, 14, 16, or 20 in. deep. They are tapered in cross section, as shown in Fig. 18.7, generally at a slope of 1 to 12, to facilitate removal. Any joist width can be obtained by varying the width of the soffit (bottom) form. Tapered end pans are used where it is desired to obtain a wider joist near the end supports, such as may be required for high shear or negative bending moment. After the concrete has hardened, the steel pans are removed for reuse.

According to ACI Code 8.11.2, ribs must not be less than 4 in. wide and may not have a depth greater than 3.5 times the minimum web width. (For easier bar placement and placement of concrete, a minimum web width of 5 in. is desirable.) The clear spacing between ribs (determined by the pan width) must not exceed 30 in. The slab thickness over the top of the pans must not be less than one-twelfth of the clear distance between ribs, nor less than 2 in., according to ACI Code 8.11.6. Table 18.1 gives unit weights, in terms of psf of floor surface, for common combinations of joist width and depth, slab thickness, and form width.

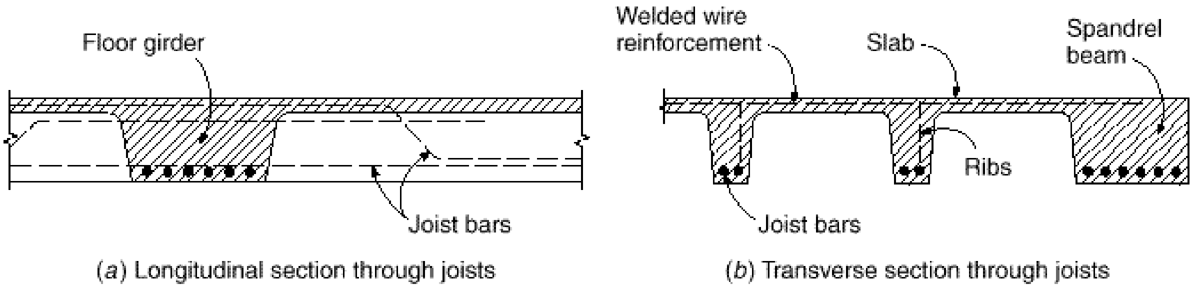


FIGURE 18.7
One-way joist floor cross sections: (*a*) cross section through supporting girder showing ends of joists; (*b*) cross section through typical joists.

TABLE 18.1
Weight of one-way joist floor systems

3 in. Top Slab			$4\frac{1}{2}$ in. Top Slab		
Depth of Pan Form, in.	Width of Joist + Pan Form, in.	Weight, psf	Depth of Pan Form, in.	Width of Joist + Pan Form, in.	Weight, psf
8	5 + 20	60	8	5 + 20	79
8	5 + 30	54	8	5 + 30	72
10	5 + 20	67	10	5 + 20	85
10	5 + 30	58	10	5 + 30	77
12	5 + 20	74	12	5 + 20	92
12	5 + 30	63	12	5 + 30	82
14	5 + 30	68	14	5 + 30	87
14	6 + 30	72	14	6 + 30	91
16	6 + 30	78	16	6 + 30	97
16	7 + 30	83	16	7 + 30	101
20	6 + 30	91	20	6 + 30	109
20	7 + 30	96	20	7 + 30	115

Source: Adapted from Ref. 18.2.

Reinforcement for the joists usually consists of two bars in the positive bending region, with one bar discontinued where no longer needed or bent up to provide a part of the negative steel requirement over the supporting girders. Straight top bars are added over the support to provide for the negative bending moment. According to ACI Code 7.13.2, at least one bottom bar must be continuous over the support, or at noncontinuous supports, terminated in a standard hook, as a measure to improve structural integrity in the event of major structural damage.

ACI Code 7.7.1 permits a reduced concrete cover of $\frac{3}{4}$ in. to be used for joist construction, just as for slabs. The thin slab (top flange) is reinforced mainly for temperature and shrinkage stresses, using welded wire reinforcement or small bars placed at right angles to the joists. The area of this reinforcement is usually 0.18 percent of the gross cross section of the concrete slab.

One-way joists are generally proportioned with the concrete providing all of the shear strength, with no stirrups used. A 10 percent increase in V_c above the value given by Eq. (4.12*a*) or (4.12*b*) is permitted for joist construction, according to ACI Code 8.11.8, based on the possibility of redistribution of local overloads to adjacent joists.

The joists and the supporting girders are placed monolithically. Like the joists, the girders are designed as T beams. The shape of the girder cross section depends on the shape of the end pans that form the joists, as shown in Fig. 18.7*a*. If the girders are deeper than the joists, the thin concrete slab directly over the top of the pans is often neglected in the girder design, and the T beam flange thickness is taken as the full height of the joists. In the latter case, the flange width can be adjusted, as needed, by varying the placement of the end pans. The width of the web below the bottom of the joists must be at least 3 in. narrower than the flange (on either side) to allow for pan removal.

A type of one-way joist floor system has evolved known as a *joist-band system* in which the joists are supported by broad girders having the same total depth as the joists, as illustrated in Fig. 18.7. Separate beam forms are eliminated, and the same deck forms the soffit of both the joists and the girders. The simplified formwork, faster construction, level ceiling with no obstructing beams, and reduced overall height of walls, columns, and vertical utilities combine to achieve an overall reduction in cost in most cases.

In one-way joist floors, the thickness of the slab is often controlled by fire resistance requirements. For a rating of 2 hours, for example, the slab must be about $4\frac{1}{2}$ in. thick. If 20 or 30 in. pan forms are used, slab span is small and slab strength is underutilized. This has led to what is known as the *wide module joist system*, or *skip joist system* (Ref. 18.5). Such floors generally have 6 to 8 in. wide ribs that are 5 to 6 ft on centers, with a $4\frac{1}{2}$ in. top slab. These floors not only provide more efficient use of concrete in the slab, but also require less formwork labor. By ACI Code 8.11.4, wide module joist ribs must be designed as ordinary T beams, because the clear spacing between ribs exceeds the 30 in. maximum for joist construction, and the special ACI Code provisions for joists do not apply. Concrete cover for reinforcement is as required for beams, not joists, and the 10 percent increase in V_c does not apply. Often the joists in wide module systems are carried by wide beams on the column lines, the depth of which is the same as that of the joists, to form a joist-band system equivalent to that described earlier.

Useful design information pertaining to one-way joist floors, including extensive load tables, will be found in the *CRSI Handbook* (Ref. 18.2). Suggested bar details and typical design drawings are found in the *ACI Detailing Manual* (Ref. 18.3).

e. Two-Way Edge-Supported Slabs

Two-way solid slabs supported by beams on the column lines on all sides of each slab panel have been discussed in detail in Chapter 13. The perimeter beams are usually concrete cast monolithically with the slab, although they may also be structural steel, often encased in concrete for composite action and for improved fire resistance. For monolithic concrete, both the beams and the slabs are designed using the direct design method or the equivalent frame method described in Chapter 13.

Two-way solid slab systems are suitable for intermediate to heavy loads on spans up to about 30 ft. This range corresponds closely to that for beamless slabs with drop panels and column capitals, described in the following section. The latter are often preferred because of the complete elimination of obstructing beams below the slab.

For lighter loads and shorter spans, a two-way solid slab system has evolved in which the column-line beams are wide and shallow, such that a cross section through the floor in either direction resembles the slab-band shown earlier in Fig. 18.4. The result is a two-way slab-band floor that, from below, appears as a paneled ceiling. Advantages are similar to those given earlier for one-way slab-band floors and for joist-band systems.

f. Beamless Flat Slabs with Drop Panels or Column Capitals

By suitably proportioning and reinforcing the slab, it is possible to eliminate supporting beams altogether. The slab is supported directly on the columns. In a rectangular or square region centered on the columns, the slab may be thickened and the column tops flared, as shown in Fig. 18.8. The thickened slab is termed a *drop panel* and the column flare is referred to as a *column capital*. Both of these serve a double purpose: they increase the shear strength of the floor system in the critical region around the

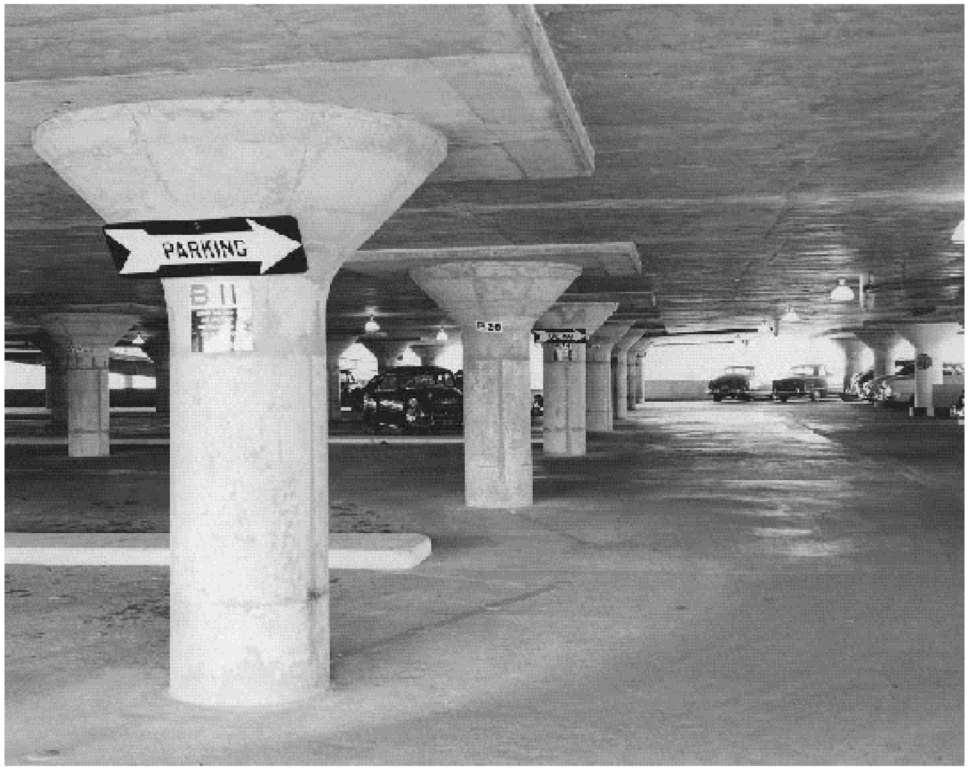


FIGURE 18.8
Flat slab garage floor with both drop panels and column capitals. (*Courtesy of Portland Cement Association.*)

column and they provide increased effective depth for the flexural steel in the region of high negative bending moment over the support. Beamless systems with drop panels or column capitals or both are termed *flat slab systems* (although almost all slabs in structural engineering practice are "flat" in the usual sense of the word), and are differentiated from flat plate systems, with absolutely no projections below the slab, which are described in the following section.

In general, flat slab construction is economical for live loads of 100 psf or more and for spans up to about 30 ft. It is widely used for storage warehouses, parking garages, and below-grade structures carrying heavy earth-fill loads, for example. For lighter loads such as in apartment houses, hotels, and office buildings, flat plates (Section 18.2g) or some form of joist construction (Sections 18.2d and h) will usually prove less expensive. For spans longer than about 30 ft, beams and girders are used because of the greater stiffness of that form of construction.

Flat slabs may be designed by the direct design method or the equivalent frame method, both described in detail in Chapter 13, or the strip method described in Chapter 15.

g. Flat Plate Slabs

A flat plate floor is essentially a flat slab floor with the drop panels and column capitals omitted, so that a floor of uniform thickness is carried directly by prismatic columns. Flat plate floors have been found to be economical and otherwise advantageous for such uses as apartment buildings, as shown in Fig. 18.9, where the spans are

FIGURE 18.9
Flat plate floor construction. (*Courtesy of Portland Cement Association.*)

moderate and loads relatively light. The construction depth for each floor is held to the absolute minimum, with resultant savings in the overall height of the building. The smooth underside of the slab can be painted directly and left exposed for ceiling, or plaster can be applied to the concrete. Minimum construction time and low labor costs result from the very simple formwork.

Certain problems associated with flat plate construction require special attention. Shear stresses near the columns may be very high, requiring the use of special forms of slab reinforcement there. The transfer of moments from slab to columns may further increase these shear stresses and requires concentration of negative flexural steel in the region close to the columns. Both these problems are treated in detail in Chapter 13. At the exterior columns, where such shear and moment transfer may cause particular difficulty, the design is much improved by extending the slab past the column in a short cantilever.

Some flat plate buildings are constructed by the lift slab method, shown in Fig. 18.10. A casting bed (often doubling as the ground-floor slab) is placed, steel columns are erected and braced, and at ground level successive slabs, which will later become the upper floors, are cast. A membrane or sprayed parting agent is laid down between successive pours so that each slab can be lifted in its turn, starting with the top. Jacks placed atop the columns are connected to threaded rods extending down the faces of the columns and connecting, in turn, to lifting collars embedded in the slabs, as shown in Fig. 13.24*d*. When a slab is in its final position, shear plates are welded to the column below the lifting collar, or other devices are used to transfer the vertical slab reaction. Lifting collars such as those shown in Fig. 13.24*d*, in addition to providing anchorage for the lifting rods, serve to increase the effective size of the support for the slab and consequently improve the shear strength of the slab. The successful erection

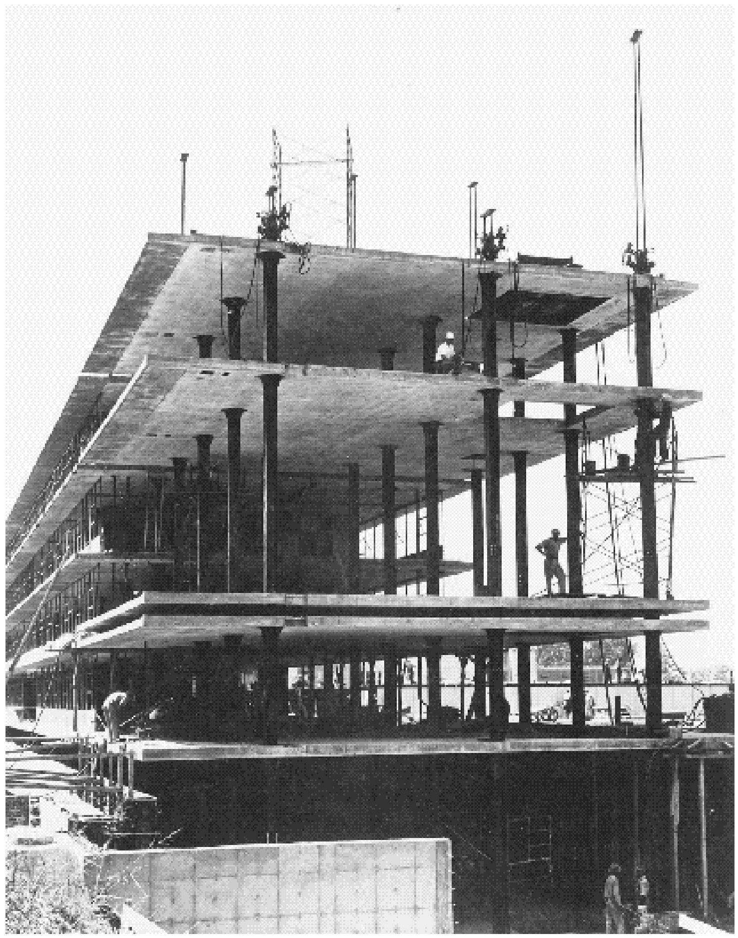


FIGURE 18.10
Lift slab construction used with flat plate floors; student dormitory at Clemson University, South Carolina.

of structures using the lift slab method requires precise control of the lifting operation at all times, because even slight differences in level of the support collars may drastically change moments and shears in the slab, possibly leading to reversal of loading. Catastrophic accidents have resulted from failure to observe proper care in lifting or to provide adequate lateral bracing for the columns. As a result of these accidents, this method of construction is used only by specialized contractors.

h. Two-Way Joist Floors

As in one-way floor systems, the dead weight of two-way slabs can be reduced considerably by creating void spaces in what would otherwise be a solid slab. For the most part, the concrete removed is in tension and ineffective, so the lighter floor has virtually the same structural characteristics as the corresponding solid floor. Voids are usually formed using dome-shaped steel pans that are removed for reuse after the slab has hardened. Forms are placed on a plywood platform as shown in Fig. 18.11. Note in the figure that domes have been omitted near the columns to obtain a solid slab in the region of negative bending moment and high shear. The lower flange of each dome contacts that of the adjacent dome, so that the concrete is cast entirely against a metal

FIGURE 18.11
Two-way joist floor under construction with steel dome forms. (*Courtesy of Ceco Corporation.*)

surface, resulting in an excellent finished appearance of the slab. A wafflelike appearance (these slabs are sometimes called waffle slabs) is imparted to the underside of the slab, which can be featured to architectural advantage, as shown in Fig. 18.12.

Two-way joist floors are designed following the usual procedures for two-way solid slab systems, as presented in Chapter 13, with the solid regions at the columns considered as drop panels. Joists in each direction are divided into column strip joists and middle strip joists, the former including all joists that frame into the solid head. Each joist rib usually includes two bars for positive-moment resistance, and one may be discontinued where no longer required. Negative steel is provided by separate straight bars running in each direction over the columns.

In design calculations, the self-weight of two-way joist floors is considered to be uniformly distributed, based on an equivalent slab of uniform thickness having the same volume of concrete as the actual ribbed slab. Equivalent thicknesses and weights are given in Table 18.2 for standard 30 and 19 in. pans of various depths and for either a 3 in. top slab or $4\frac{1}{2}$ in. top slab, based on normal-weight concrete (150 lb/ft^3).

18.3 PANEL, CURTAIN, AND BEARING WALLS

As a general rule, the exterior walls of a reinforced concrete building are supported at each floor by the skeleton framework, their only function being to enclose the building. Such walls are called *panel walls*. They may be made of concrete (often precast), concrete block, brick, tile blocks, or insulated metal panels. The latter may be faced with aluminum, stainless steel, or a porcelain-enamel finish over steel, backed by insulating material and an inner surface sheathing. The thickness of each of these types of panel walls will vary according to the material, type of construction, climatological conditions, and the building requirements governing the particular locality in which the construction takes place.

FIGURE 18.12
Regency House Apartments, San Antonio, with cantilevered two-way joist slab plus integral beams on column lines.

TABLE 18.2
Equivalent slab thickness and weight of two-way joist floor systems

	3 in. Top Slab		$4\frac{1}{2}$ in. Top Slab	
Depth of Pan Form, in.	Equivalent Uniform Thickness, in.	Weight, psf	Equivalent Uniform Thickness, in.	Weight, psf
36 in. Module (30 in. pans plus 6 in. ribs)				
8	5.8	73	7.3	92
10	6.7	83	8.2	102
12	7.4	95	9.1	114
14	8.3	106	9.9	120
16	9.1	114	10.6	133
20	10.8	135	12.3	154
24 in. Module (19 in. pans plus 5 in. ribs)				
8	6.8	85	8.3	103
10	7.3	91	8.8	111
12	8.6	107	10.1	126

Source: Adapted from Ref. 18.2.

The pressure of the wind is usually the only load that is considered in determining the structural thickness of a wall panel, although in some cases exterior walls are used as diaphragms to transmit forces caused by horizontal loads down to the building foundations.

Curtain walls are similar to panel walls except that they are not supported at each story by the frame of the building, but are self-supporting. However, they are often anchored to the building frame at each floor to provide lateral support.

A *bearing wall* may be defined as one that carries any vertical load in addition to its own weight. Such walls may be constructed of stone masonry, brick, concrete block, or reinforced concrete. Occasional projections or pilasters add to the strength of the wall and are often used at points of load concentration. In small commercial buildings, bearing walls may be used with economy and expediency. In larger commercial and manufacturing buildings, when the element of time is an important factor, the delay necessary for the erection of the bearing wall and the attendant increased cost of construction often dictate the use of some other arrangement.

18.4 SHEAR WALLS

Horizontal forces acting on buildings, e.g., those due to wind or seismic action, can be resisted by different means. Rigid-frame resistance of the structure, augmented by the contribution of ordinary masonry walls and partitions, can provide for wind loads in many cases. However, when heavy horizontal loading is likely, such as would result from an earthquake, reinforced concrete shear walls are used. These may be added solely to resist horizontal forces, or concrete walls enclosing stairways or elevator shafts may also serve as shear walls.

Figure 18.13 shows a building with wind or seismic forces represented by arrows acting on the edge of each floor or roof. The horizontal surfaces act as deep beams to transmit loads to vertical resisting elements A and B. These shear walls, in turn, act as cantilever beams fixed at their base to carry loads down to the foundation. They are

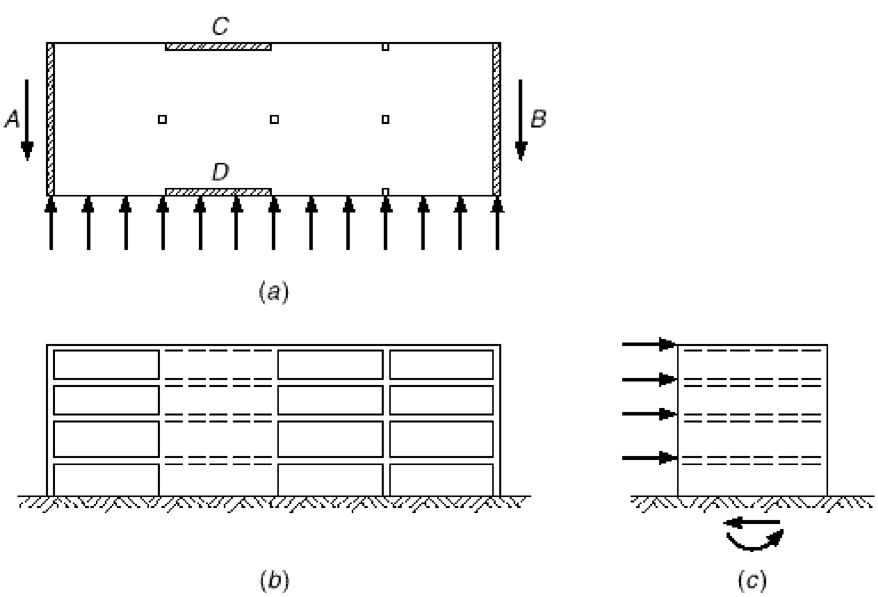


FIGURE 18.13
Building with shear walls subject to horizontal loads: (*a*) typical floor; (*b*) front elevation; (*c*) end elevation.

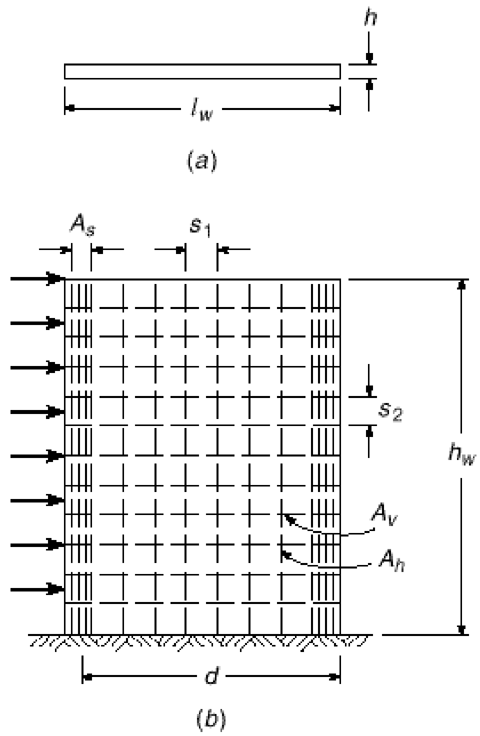


FIGURE 18.14
Geometry and reinforcement of a typical shear wall: (*a*) cross section; (*b*) elevation.

subjected to (1) a variable shear, which reaches a maximum at the base, (2) a bending moment, which tends to cause vertical tension near the loaded edge and compression at the far edge, and (3) a vertical compression due to ordinary gravity loading from the structure. For the building shown, additional shear walls *C* and *D* are provided to resist loads acting in the long direction of the structure.

Shear is apt to be critical for walls with a relatively low ratio of height to length. High shear walls are controlled mainly by flexural requirements.

Figure 18.14 shows a typical shear wall with height h_w, length l_w, and thickness h. It is assumed to be fixed at its base and loaded horizontally along its left edge. Vertical flexural reinforcement of area A_s is provided at the left edge, with its centroid a distance d from the extreme compression face. To allow for reversal of load, identical reinforcement is provided along the right edge. Horizontal shear reinforcement with area A_v at spacing s_2 is provided, as well as vertical shear reinforcement with area A_h at spacing s_1. Such distributed steel is normally placed in two layers, parallel to the faces of the wall.

The design basis for shear walls, according to ACI Code 11.10, is of the same general form as that used for ordinary beams:

$$V_u \leq \cdot\ V_n \tag{18.1}$$

where

$$V_n = V_c + V_s \tag{18.2}$$

Based on tests (Refs. 18.6 and 18.7), an upper limit has been established on the nominal shear strength of walls:

$$V_n \leq 10 \cdot \sqrt{f_c'}\, hd \tag{18.3}$$

In this and all other equations pertaining to the design of shear walls, the distance d is taken equal to $0.8l_w$. A larger value of d, equal to the distance from the extreme compression face to the center of force of all reinforcement in tension, may be used when determined by a strain compatibility analysis.

The value of V_c, the nominal shear strength provided by the concrete, may be based on the usual equations for beams, according to ACI Code 11.10.5. For walls subject to vertical compression,

$$V_c = 2\sqrt{f'_c}hd \tag{18.4}$$

and for walls subject to vertical tension N_u,

$$V_c = 2\left(1 + \frac{N_u}{500A_g}\right)\sqrt{f'_c}hd \tag{18.5}$$

Here, N_u is the factored axial load in pounds, taken negative for tension, and A_g is the gross area of horizontal concrete section in square inches. Alternately, the value of V_c may be based on a more detailed calculation, as the lesser of

$$V_c = 3.3\sqrt{f'_c}hd + \frac{N_u d}{4l_w} \tag{18.6}$$

or

$$V_c = \left[0.6\sqrt{f'_c} + \frac{l_w\left(1.25\sqrt{f'_c} + 0.2N_u/l_w h\right)}{M_u/V_u - l_w/2}\right]hd \tag{18.7}$$

where N_u is negative for tension as before. Equation (18.6) corresponds to the occurrence of a principal tensile stress of approximately $4\sqrt{f'_c}$ at the centroid of the shear-wall cross section. Equation (18.7) corresponds approximately to the occurrence of a flexural tensile stress of $6\sqrt{f'_c}$ at a section $l_w/2$ above the section being investigated. Thus, the two equations predict, respectively, web-shear cracking and flexure-shear cracking. When the quantity $M_u/V_u - l_w/2$ is negative, Eq. (18.7) is inapplicable. According to the ACI Code, horizontal sections located closer to the wall base than a distance $l_w/2$ or $h_w/2$, whichever is less, may be designed for the same V_c as that computed at a distance $l_w/2$ or $h_w/2$

When the factored shear force V_u does not exceed $\phi V_c/2$, a wall may be reinforced according to minimum requirements. When V_u exceeds $\phi V_c/2$, reinforcement for shear is to be provided according to the following requirements.

The nominal shear strength V_s provided by the horizontal wall steel is determined on the same basis as for ordinary beams:

$$V_s = \frac{A_v f_y d}{s_2} \tag{18.8}$$

where A_v = area of horizontal shear reinforcement within vertical distance s_2, in^2
s_2 = vertical distance between horizontal reinforcement, in.
f_y = yield strength of reinforcement, psi

Substituting Eq. (18.8) into Eq. (18.2), then combining with Eq. (18.1), one obtains the equation for the required area of horizontal shear reinforcement within a distance s_2:

$$A_v = \frac{(V_u - \phi V_c)s_2}{\phi f_y d} \tag{18.9}$$

The minimum permitted ratio of horizontal shear steel to gross concrete area of vertical section is

$$\rho_h = 0.0025 \tag{18.10}$$

and the maximum spacing s_2 is not to exceed $l_w/5$, $3h$, or 18 in.

Test results indicate that for low shear walls vertical distributed reinforcement is needed as well as horizontal reinforcement. Code provisions require vertical steel of area A_h within a spacing s_1, such that the ratio of vertical steel to gross concrete area of horizontal section will be not less than

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_h - 0.0025) \tag{18.11}$$

nor less than 0.0025. However, the vertical reinforcement ratio need not be greater than the required horizontal reinforcement ratio. The spacing of the vertical bars is not to exceed $l_w/3$, $3h$, or 18 in.

Walls may be subject to flexural tension due to overturning moment, even when the vertical compression from gravity loads is superimposed. In many but not all cases, vertical steel is provided, concentrated near the wall edges, as shown in Fig. 18.14. The required steel area can be found by the usual methods for beams.

The dual function of the floors and roofs in buildings with shear walls should be noted. In addition to resisting gravity loads, they must act as deep beams spanning between shear-resisting elements. Because of their proportions, both shearing and flexural stresses are usually quite low. According to ACI Code 9.2.1, the load factor for live load drops to 1.0 when wind or earthquake effects are combined with the effects of gravity loads. Consequently, floor and roof reinforcement designed for gravity loads can usually serve as reinforcement for horizontal beam action also, with no increase in bar areas.

ACI Code 10.11.1 permits walls with height-to-length ratios not exceeding 2.0 to be designed using strut-and-tie models (Chapter 10). The minimum shear reinforcement criteria of Eqs. (18.9) through (18.11) and the maximum spacing limits for s_1 and s_2 must be satisfied.

There are special considerations and requirements for the design of reinforced concrete walls in structures designed to resist forces associated with seismic motion. These are based on design for energy dissipation in the nonlinear range of response. This subject will be treated separately, in Chapter 20.

18.5 Precast Concrete for Buildings

The earlier sections in this chapter have emphasized cast-in-place reinforced concrete structures. Construction of these structures requires a significant amount of skilled on-site labor. There is, however, another class of concrete construction for which the members are manufactured off site in precasting yards, under factory conditions, and subsequently assembled on site, a process that provides significant advantages in terms of economy and speed of construction.

Precast concrete construction involves the mass production of repetitive and often standardized units: columns, beams, floor and roof elements, and wall panels. On large jobs, precasting yards are sometimes constructed on or adjacent to the site. More frequently, these yards are stationary regional enterprises that supply precast members to sizable areas within reasonable shipping distances, on the order of 200 mi.

Advantages of precast construction include less labor per unit because of mechanized series production; use of unskilled local labor, in contrast to skilled mobile construction labor; shorter construction time because site labor primarily involves only foundation construction and connecting the precast units; better quality control and higher concrete strength that is achievable under factory conditions; and greater independence of construction from weather and season. Disadvantages are the greater cost of transporting precast units, as compared with transporting materials, and the additional technical problems and costs of site connections of precast elements.

Precast construction is used in all major types of structures: industrial buildings, residential and office buildings, halls of sizable span, bridges, stadiums, and prisons. Precast members frequently are prestressed in the casting yard. In the context of the present chapter, it is irrelevant whether a precast member is also prestressed. Discussion is focused on types of precast members and precast structures and on methods of connection; these are essentially independent of whether the desired strength of the member was achieved with ordinary reinforcement or by prestressing. A broader discussion of precast construction, which includes planning, design, materials, manufacturing, handling, construction, and inspection, will be found in Refs. 18.8 and 18.9. ACI Code Chapter 16 is dedicated to precast concrete.

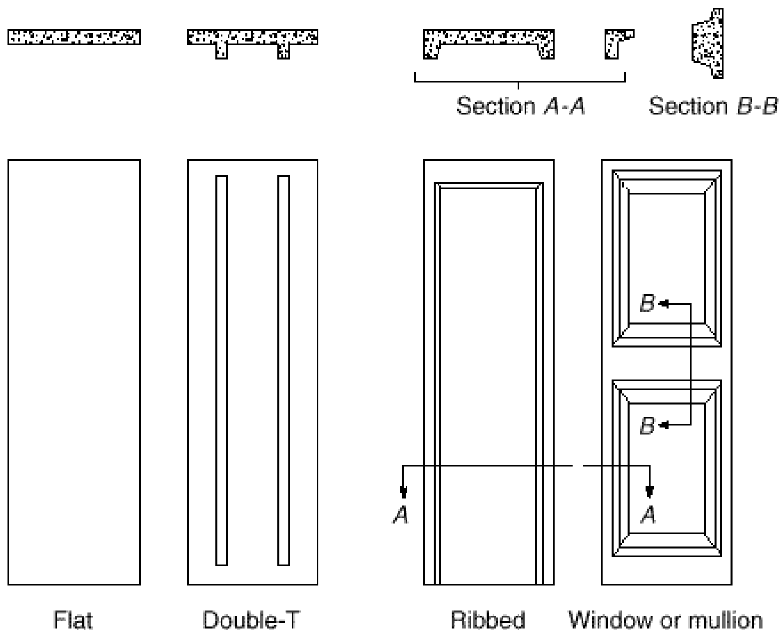
a. Types of Precast Members

A number of types of precast units are in common use. Though most are not formally standardized, they are widely available, with minor local variations. At the same time, the precasting process is sufficiently adaptable for special shapes developed for particular projects to be produced economically, provided that the number of repetitive units is sufficiently large. This is particularly important for exterior wall panels, which permit a wide variety of architectural treatments.

Wall panels are made in a considerable variety of shapes, depending on architectural requirements. The most frequent four shapes are shown in Fig. 18.15. These units are produced in one to four-story-high sections and up to 8 ft in width. They are used either as curtain walls attached to columns and beams or as bearing walls. To improve thermal insulation, sandwich panels are used that consist of an insulation core (e.g., foam glass, glass fiber, or expanded plastic) between two layers of normal or lightweight concrete. The two layers must be adequately interconnected through the core to act as one unit. A variety of surface finishes can be produced through the use of special exposed aggregates or of colored cement, sometimes employed in combination. The special design problems that arise in load-bearing wall panels, such as tilt-up construction, are discussed in Ref. 18.10.

Stresses in wall panels are frequently more severe in handling and during erection than in the finished structure, and the design must provide for these temporary conditions. Also, control of cracking is of greater importance in wall panels than in other precast units, for appearance more than for safety. To control cracking, the maximum tensile stress in the concrete, calculated by straight-line theory, should not exceed the modulus of rupture of the particular concrete with an adequate margin of safety. ACI Committee 533 (Ref. 18.11) recommends that tensile stresses for normal-weight concrete be limited to $5\sqrt{f_c'}$ under the effects of form removal, handling, transportation, impact, and live load. Maximum tensile stresses equal to 75 and 85 percent of this value are recommended for all-lightweight and sand-lightweight concrete, respectively. A wealth of information on precast wall panels is found in Refs. 18.9 and 18.11.

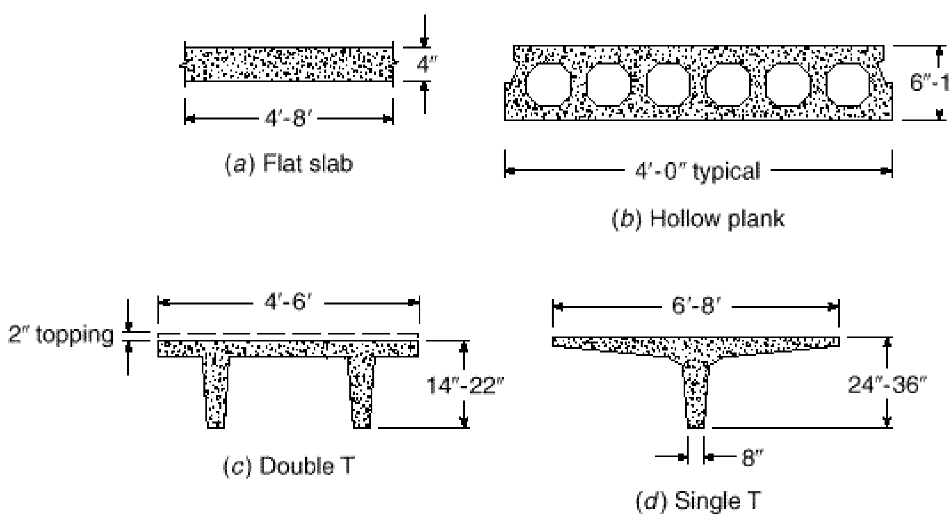
FIGURE 18.15
Precast concrete wall panels.



Roof and floor elements are made in a wide variety of shapes adapted to specific conditions, such as span lengths, magnitude of loads, desired fire ratings, and appearance. Figure 18.16 shows typical examples of the most common shapes, arranged in approximate order of increasing span length, even though the spans covered by the various configurations overlap widely.

Flat slabs (Fig. 18.16*a*) are usually 4 in. thick, although they are used as thin as $2\frac{1}{2}$ in. when continuous over several spans, and are produced in widths of 4 to 8 ft and in lengths up to 36 ft. Depending on the magnitude of loads and on deflection limitations, they are used over roof and floor spans ranging from 8 to about 22 ft. For lower

FIGURE 18.16
Precast floor and roof elements.



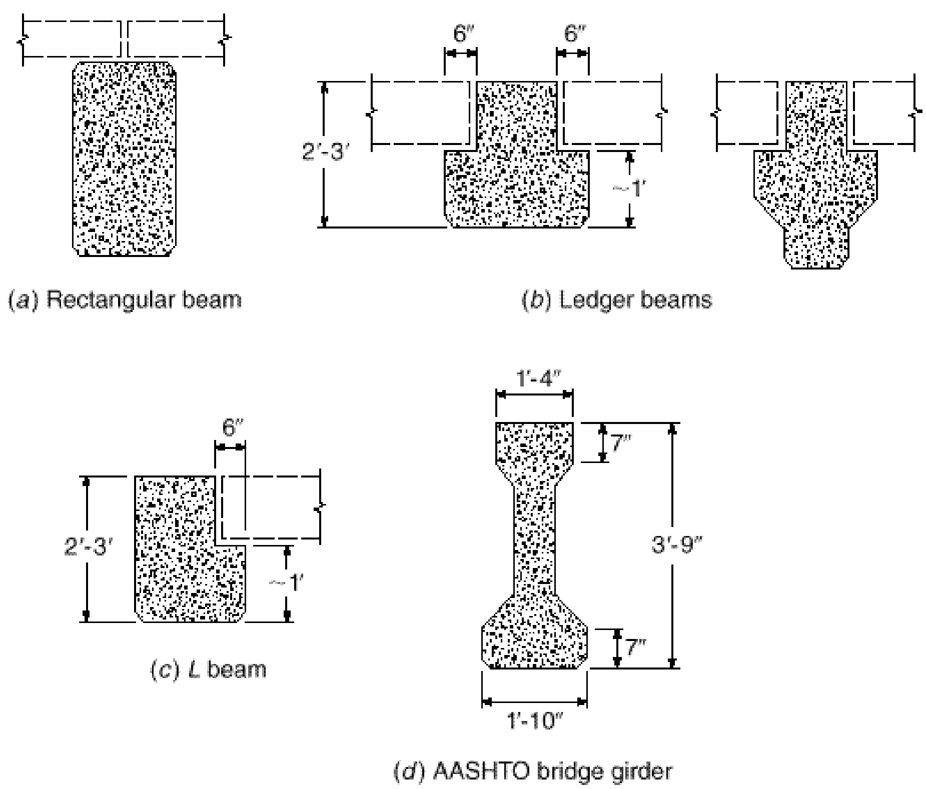
weight and better insulation and to cover longer spans, *hollow-core planks* (Fig. 18.16*b*) with a variety of shapes are used. Some of these are made by extrusion in special machines. Depths range from 6 to 12 in., with widths of 3 or 4 ft. Again depending on load and deflection requirements, they are used on roof spans from about 16 to 34 ft and on floor spans from 12 to 26 ft, which can be augmented to about 30 ft if a 2 in. topping is applied to act monolithically with the hollow plank.

For longer spans, *double T* members (Fig. 18.16*c*) are the most widely used shapes. Usual depths are from 14 to 22 in. They are used on roof spans up to 120 ft. When used as floor members, a concrete topping of at least 2 in. is usually applied to act monolithically with the precast members for spans up to about 50 ft, depending on load and deflection requirements. Finally, *single T* members are available in dimensions shown in Fig. 18.16*d*, mostly used for roof spans up to 100 ft and more.

For all of these units, the member itself or its flange constitutes the roof or floor slab. If the floor or roof proper is made of other material (e.g., plywood, gypsum, and plank), it can be supported on *precast joists* in a variety of shapes for spans from about 15 to 60 ft. Reference 18.9 addresses the design of both reinforced and prestressed concrete floor and roof units.

The shape of *precast beams* depends chiefly on the manner of framing. If floor and roof members are supported on top of the beams, these are mostly rectangular in shape (Fig. 18.17*a*). To reduce total depth of floor and roof construction, the tops of beams are often made flush with the top surface of the floor elements. To provide bearing, the beams are then constructed as ledger beams (Fig. 18.17*b*) or L beams (Fig.

FIGURE 18.17
Precast beams and girders.



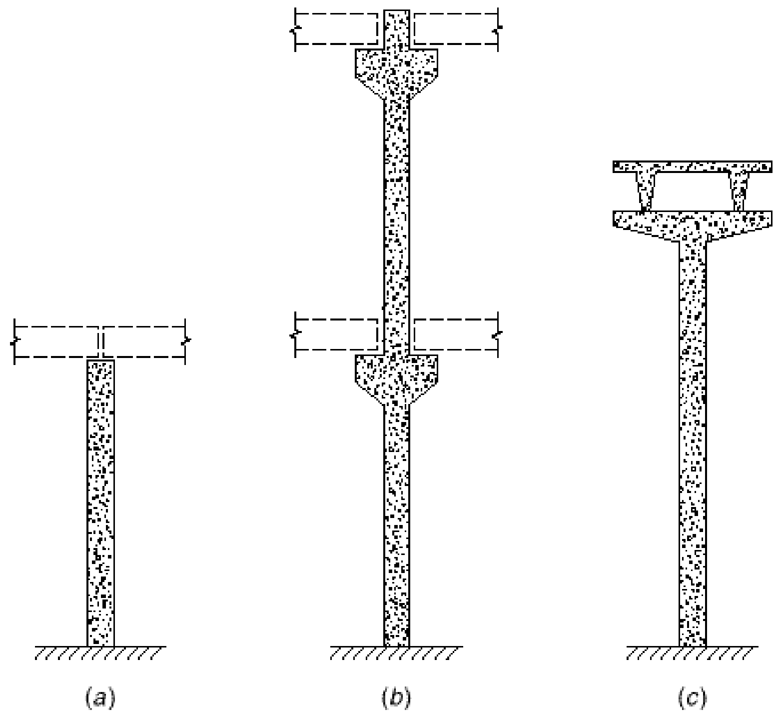


FIGURE 18.18
Precast concrete columns.

18.17*c*). Although these shapes pertain to building construction, precast beams or girders are also frequently used in highway bridges. As an example, Fig. 18.17*d* shows one of the various AASHTO bridge girders, so named because they were developed by the American Association of State Highway and Transportation Officials.

If *precast columns* of single-story height are used so that the beams rest on top of the columns, simple prismatic columns are employed, which are available in sizes from about 12 × 12 to 24 × 24 in. (Fig. 18.18*a*). In this case, the beams are usually made continuous over the columns. Alternatively, in multistory construction, the columns can be made continuous for up to about six stories. In this case, integral brackets are frequently used to provide a bearing for the beams, as shown in Fig. 18.18*b* (see also Section 18.6b). Occasionally, T columns are used for direct support of double T floor members without the use of intermediate beams (Fig. 18.18*c*).

Figures 18.19 to 18.27 illustrate some of the many ways in which precast members have been used. Figure 18.19 shows a long-span single T girder being lowered into place atop a precast rectangular beam, which in turn rests on a precast rectangular column. The photograph in Fig. 18.20 was taken in a precasting yard producing a variety of L, T, and rectangular shapes. Figure 18.21 shows symmetrical precast I beams, such as are used both for buildings and bridges. The projecting stirrup bars along the top flange will provide secure interlock between the precast beams and a cast-in-place slab added later, ensuring composite action. Figure 18.22 shows a multistory parking garage in which three-story precast columns support L-section and inverted T-section girders. The girders, in turn, carry 60 ft span prestressed single T beams, which provide the deck surface.

Figure 18.23 demonstrates that unusual architectural designs can be realized in precast concrete, as in this all-precast administration building. Wall panels are used to

FIGURE 18.19
Long-span precast single T girder used with precast beams and columns.

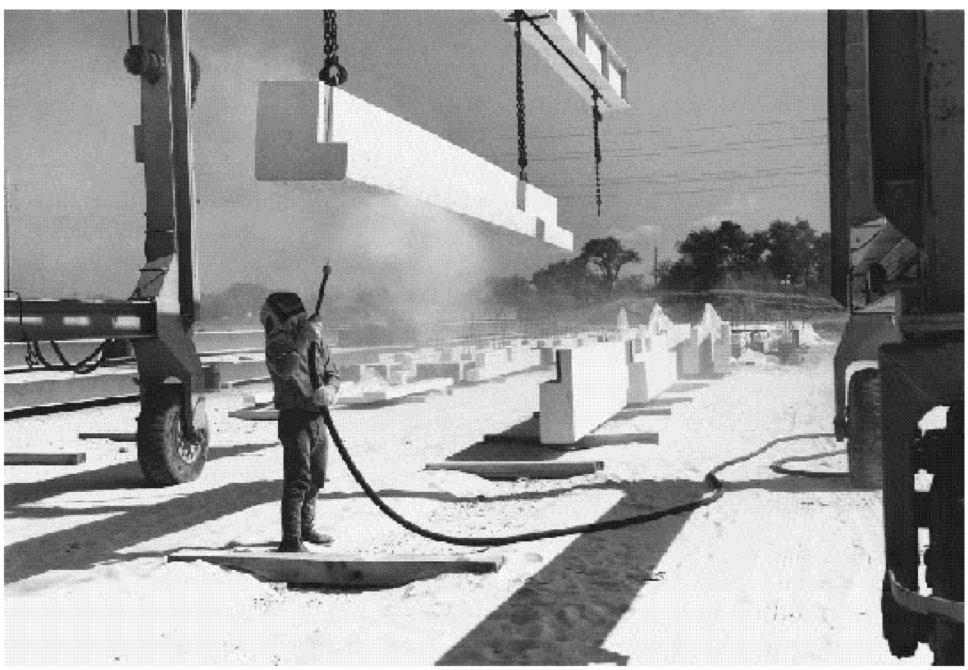


FIGURE 18.20
Precast L beam.

produce a curved facade. Wedge-shaped repetitive floor units span freely from the exterior facade to the interior curved beam and column framework. In the insurance building shown in Fig. 18.24, 44 in. deep precast girders span 99 ft between exterior walls supported on four points each and provide six floors of office space entirely free

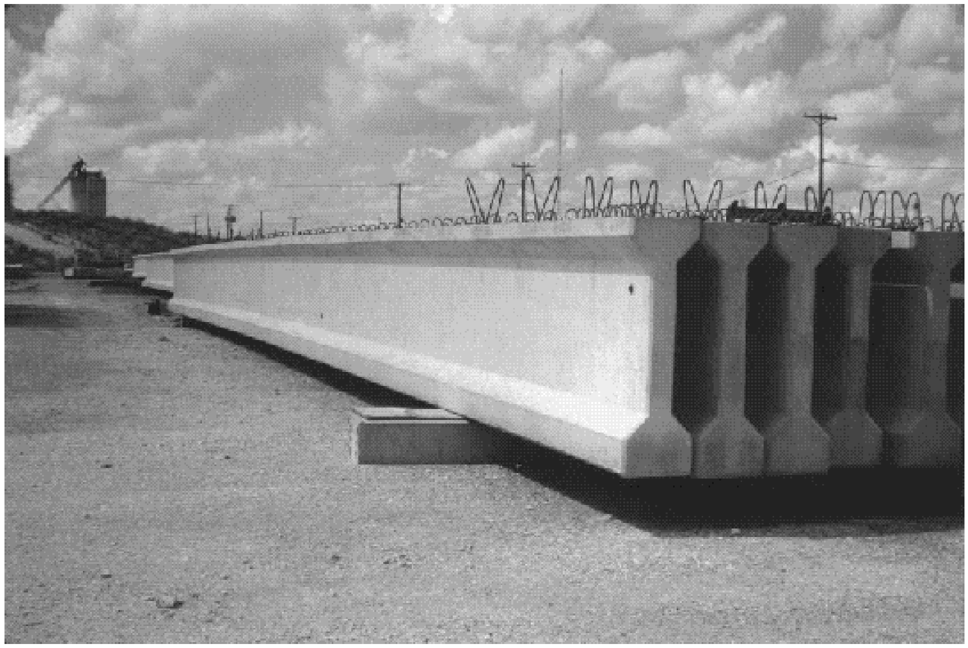


FIGURE 18.21
Precast I beams designed for composite action with a deck slab to be cast in place.

FIGURE 18.22
Precast parking garage at Cornell University.

of interior supports. The convention headquarters of Fig. 18.25 combines cast-in-place frames and floor slabs with precast double T roof beams and precast wall panels of special design. Figure 18.26 shows a 21-story hotel under construction, which, except for the service units, consists entirely of box-shaped, room-sized modules completely

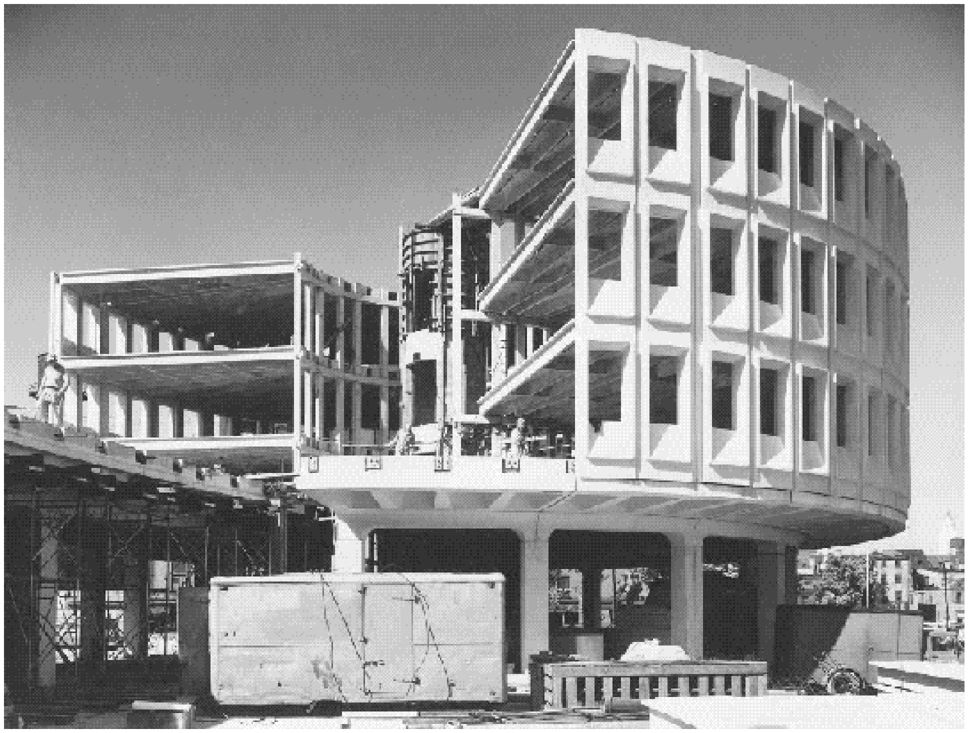


FIGURE 18.23
All precast administration building. (*Courtesy of Portland Cement Association.*)

FIGURE 18.24
Precast girders with 99 ft span and 44 in. depth for a column-free interior. (*Courtesy of Portland Cement Association.*)

FIGURE 18.25
Precast roof and wall panels combined with cast-in-place frames and floor slabs. *(Courtesy of Portland Cement Association.)*

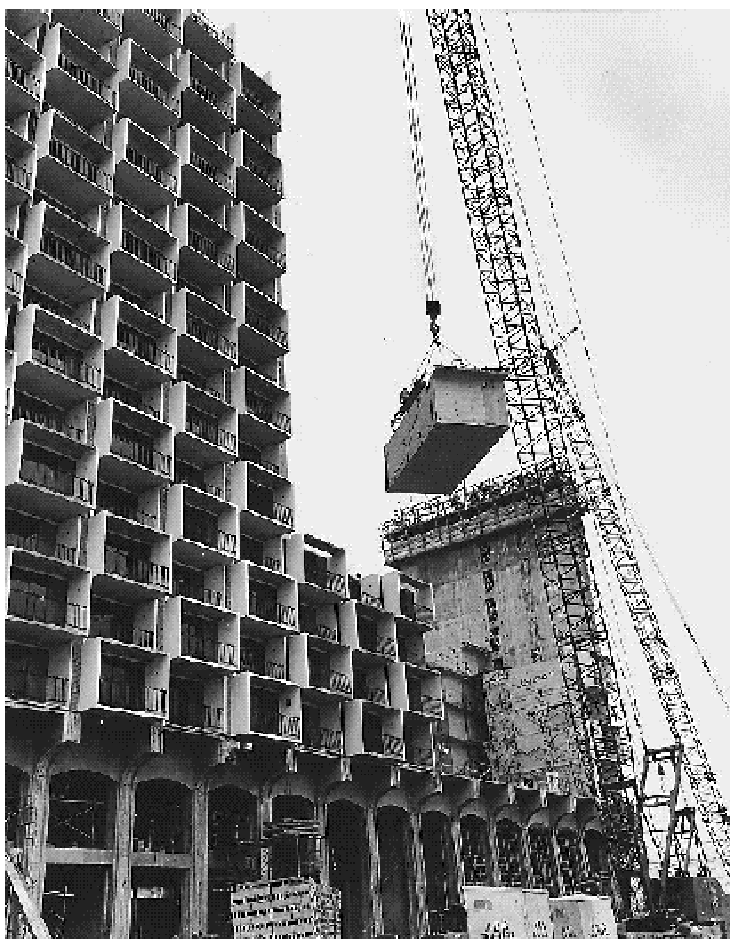


FIGURE 18.26
Precast room-sized modules for a 21-story hotel. *(Courtesy of Portland Cement Association.)*

FIGURE 18.27
Steel framing combined with precast concrete floor planks for an 8-story hotel. (*Courtesy of Bethlehem Steel Co.*)

prefabricated and stacked on top of each other. Abroad, such precast modules, with plumbing, wiring, and heating preinstalled, are widely used for multistory apartment buildings as an alternative to making similar apartment structures in precast wall, roof, and floor panels, which are more easily shipped but less easily erected than box-shaped modules.

Finally, Fig. 18.27 shows an example of the frequent combined use of structural steel with precast concrete. In this case, the framing of an eight-story hotel was done using bolted structural steel, while precast concrete floor and roof planks and precast wall panels were used for all other main structural components. This type of construction is economical for 6 to 12-story buildings, where it provides savings in both cost and construction time. It is one example of the increasingly important combined use of various structural materials and methods.

b. Connections

Cast-in-place reinforced concrete structures, by their very nature, tend to be monolithic and continuous. Connections, in the sense of joining two hitherto separate pieces, rarely occur in that type of construction. Precast structures, on the other hand, resemble steel construction in that the final structure consists of large numbers of prefabricated elements that are connected on site to form the finished structure. In both types of construction, such connections can be detailed to transmit gravity forces only, or gravity and horizontal forces, or moments in addition to these forces. In the last case, a continuous structure is obtained much as in cast-in-place construction, and connections that achieve such continuity by appropriate use of special hardware, reinforcing steel, and concrete to transmit all tension, compression, and shear stresses are sometimes called *hard* connections. In contrast, connections that transmit reactions in one direction only, analogous to rockers or rollers in steel structures, but permit a limited amount of motion to relieve other forces, such as horizontal reaction components, are sometimes known as *soft* connections (Ref. 18.12). In almost all precast connections, bearing plates or pads are used to ensure distribution and reasonable uniformity of bearing pressures. Bearing plates are made of steel, while bearing pads are made of materials such as chloroprene, fiber-reinforced polymers, and Teflon. If bearing plates are used, and the plates on two members are suitably joined by welding or other means, a hard connection is obtained in the sense that horizontal, as well as vertical, forces are transmitted. On the other hand, bearing pads transmit gravity loads but can permit sizable horizontal deformations and, thus, relieve horizontal forces.

Precast concrete structures are subject to dimensional changes from creep, shrinkage, and relaxation of prestress in addition to temperature, while in steel structures only temperature changes produce dimensional variations. In the early development of precast construction, there was a tendency to use soft connections extensively to permit these dimensional changes to occur without causing restraint forces in the members, and particularly in the connections. Subsequent experience, however, has shown that the resulting structures possess insufficient stability against lateral forces, such as high wind and, particularly, earthquake effects. Therefore, current practice emphasizes the use of hard connections that provide a high degree of continuity (Refs. 18.9 and 18.13). When designing hard connections, provisions must be made to resist the restraint forces that are caused by the previously described volume changes (Ref. 18.9). Considerable information concerning this and other matters relating to connections is found in Refs. 18.9 and 18.13.

Bearing stresses on plain concrete are limited by ACI Code 10.17.1 to $0.85\phi f_c'$, except when the supporting area is wider on all sides than the loaded area A_1. In such a case this value of the permissible bearing stress may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.0, where A_2 is the maximum portion of the supporting surface that is geometrically similar to and concentric with the loading area (see Section 16.6b).

In the design of connections, it is prudent to use load factors that exceed those required for the connected members. This is so because connections are generally subject to high stress concentrations that preclude the development of much ductility. In contrast, the members connected are likely to possess considerable ductility if designed by usual ACI Code procedures and will give warning of impending collapse if overloading should take place. In addition, imperfections in connection geometry may cause large changes in the magnitude of stresses compared with those assumed in the design.

In designing members according to the ACI Code, load factors of 1.2 and 1.6 are applied to dead and live loads, D and L respectively, to determine the required strength. When volume change effects T are considered, they are normally treated as dead load, and the factored load U is calculated from the equation $U = 1.2(D + T) + 1.6L$.

A wide variety of connection details for precast concrete building components has evolved, only a few of which will be shown here as more or less representative connections. Many additional possibilities are described fully in Refs. 18.9 and 18.13.

Column base connections are generally accomplished using steel base plates that are anchored into the precast column. Figure 18.28*a* shows a column base detail with projecting base plate. Four anchor bolts are used, with double nuts facilitating erection and leveling of the column. Typically a minimum of 2 in. of nonshrink grout is used between the top of the pier, footing, or wall and the bottom of the steel base plate. Column reinforcement is welded to the top face of the base plate. Tests have confirmed

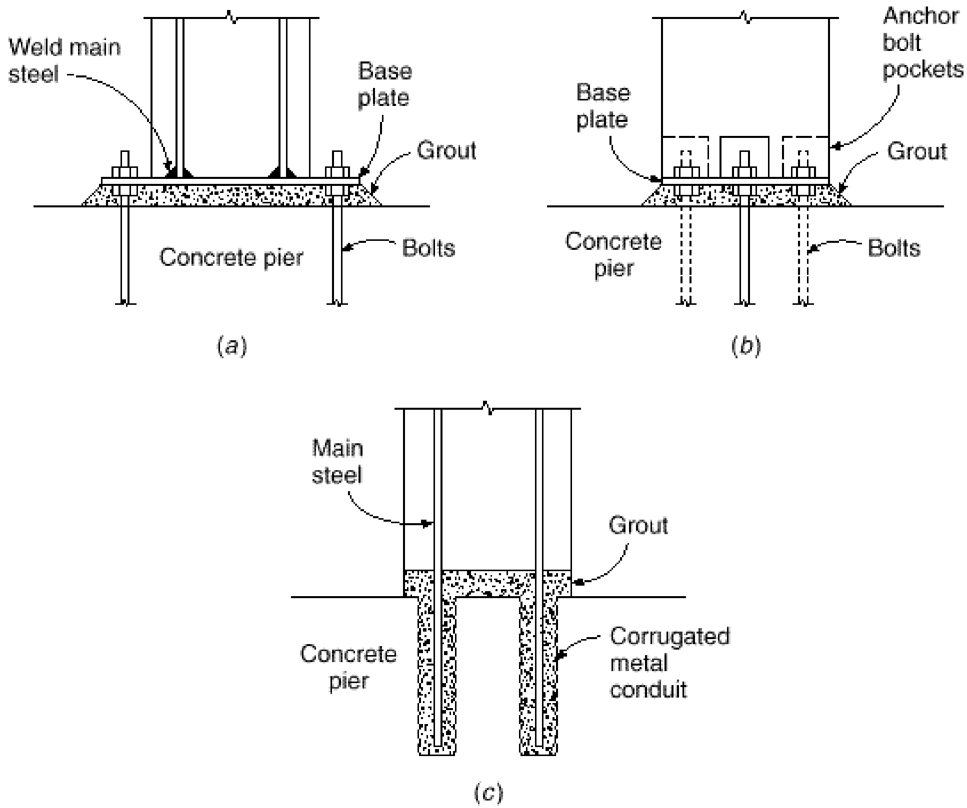


FIGURE 18.28
Column base connections.

that such column connections can transmit the full moment for which the column is designed, if properly detailed.

An alternative base detail is shown in Fig. 18.28*b*, with the dimensions of the base plate the same as, or slightly smaller than, the outside column dimensions. Anchor bolt pockets are provided, either centered on the column faces as shown, or located at the corners. Bolt pockets are grouted after the nuts are tightened. Column bars, not shown here, would be welded to the top face of the base plate as before. Figure 18.29 shows the base plate detail, similar to Fig. 18.28*b*, that was used for the precast three-story columns in the parking garage shown in Fig. 18.22.

In Fig. 18.28*c*, the main column bars project from the ends of the precast member a sufficient distance to develop their strength by bond. The projecting bars are inserted into grout-filled holes cast in the foundation when it is placed.

In all of the cases shown, confining steel should be provided around the anchor bolts in the form of closed ties. A minimum of four No. 3 (No. 10) ties is recommended, placed on 3 in. centers near the top surface of the pier or wall. Tie reinforcement in the columns should be provided as usual.

Figure 18.30 shows several *beam-to-column* connections. In all cases, rectangular beams are shown, but similar details apply to I or T beams. The figure shows only the basic geometry; and auxiliary reinforcement, anchors, and ties are omitted for the sake of clarity.

Figure 18.30*a* shows a joint detail with a concealed haunch. Well-anchored bearing angles are provided at the column seat and beam end. This type of connection may

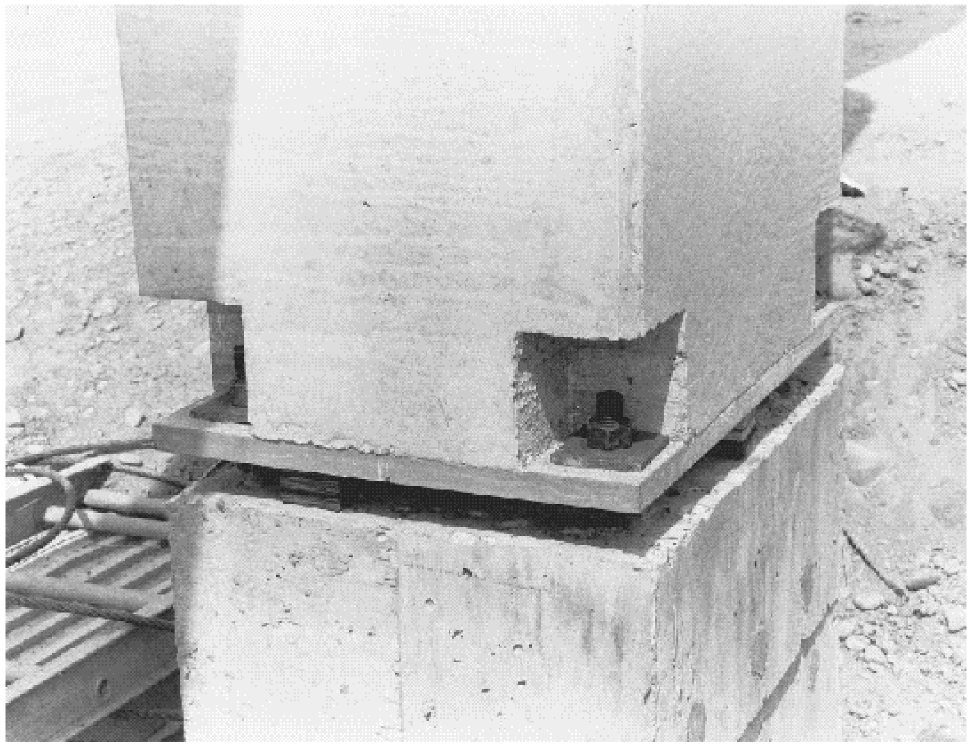


FIGURE 18.29
Detail at base of precast column of Cornell University parking garage shown in Fig. 18.22.

be used to provide vertical and horizontal reaction components, and with the addition of post-tensioned prestressing, will provide moment resistance as well.

Figure 18.30*b* shows a typical bracket, common for industrial construction where the projecting bracket is not objectionable. The seat angle is welded to reinforcing bars anchored in the column. A steel bearing plate is used at the bottom of the beam and anchored into the concrete.

The embedded steel shape in Fig. 18.30*c* is used when it is necessary to avoid projections beyond the face of the column or below the bottom of the beam. A socket is formed in casting the beam, with steel angle or plate at its top, to receive the beam stub. A steel connection can also be used in place of the bracket shown in Fig. 18.30*b*.

Finally, Fig. 18.30*d* shows a doweled connection with bars projecting from the column into holes formed in the beam ends. These are grouted after the beams are in position. This connection is popular in precast concrete construction but has little flexural capacity (Ref. 18.14).

Figure 18.31 shows several typical *column-to-column* connections. Figure 18.31*a* shows a detail using anchor bolt pockets and a double-nut system for leveling the upper column. Bolts can also be located at the center of the column faces, as shown in Fig. 18.28*b*. The detail shown in Fig. 18.31*b* permits the main steel to be lap-spliced with that in the column below. One of the many possibilities for splicing a column through a continuous beam is shown in Fig. 18.31*c*. Main reinforcing bars in both upper and lower columns should be welded to steel cap and base plates to transfer their load, and anchor bolts should be designed with the same consideration. Closely spaced ties must be provided in the columns and in this case in the beam as well, to transfer the load between columns.

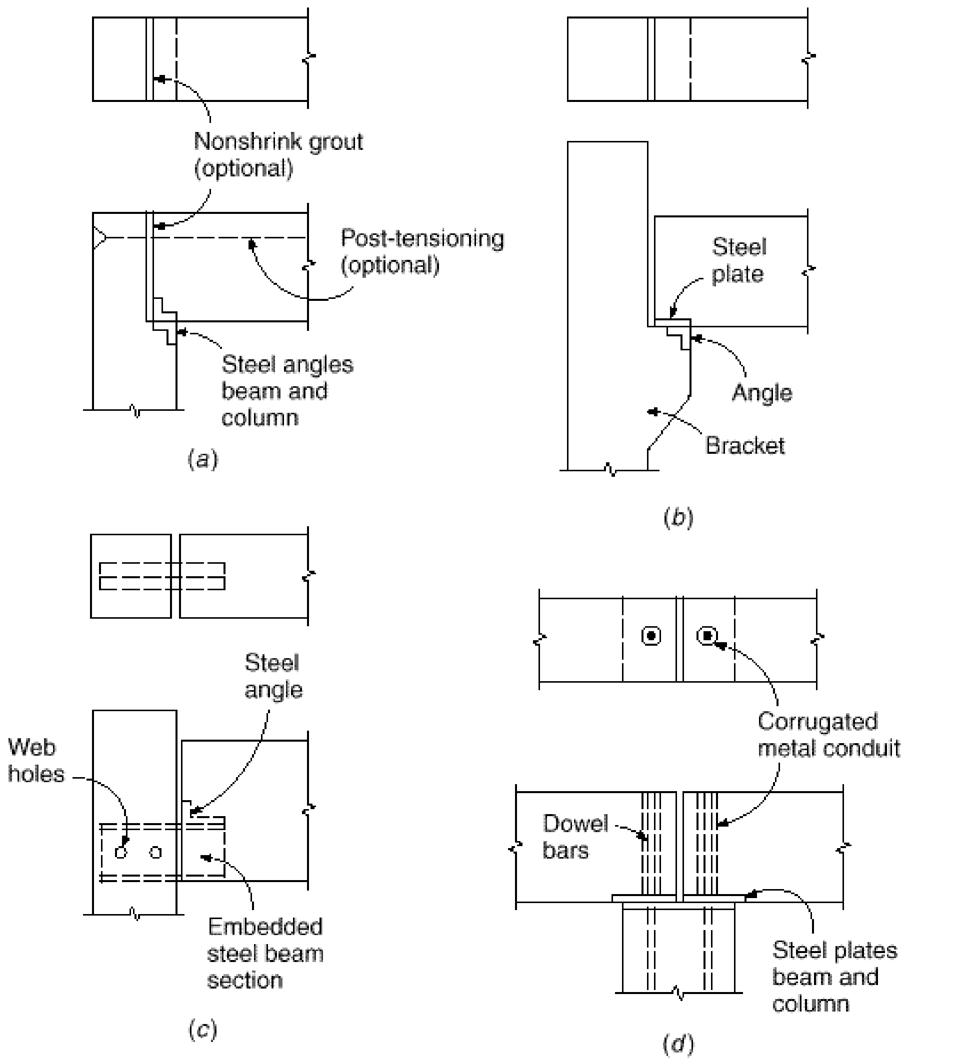


FIGURE 18.30
Beam-to-column connections.

Slab-to-beam connections generally use some variation of the detail shown in Fig. 18.32. Support is provided by an L beam (Fig. 18.32*a*) or an inverted T beam (Fig. 18.32*b*) that is flush with the top of the precast floor planks. The detail shown is sufficient if no mechanical tie is required between the precast parts. Where a positive connection is required, steel plates are set into the top of the members, suitably anchored, and short connecting plates are welded so as to attach the built-in plates.

Basic tools for the design of precast concrete connections are the *shear friction design method* described in detail in Chapter 4 and the *strut-and-tie model* introduced in Chapter 10. Example 4.6 (Section 4.9) demonstrated the use of the shear-friction approach to determining the reinforcement for the end-bearing region of a precast concrete girder. The use of both the shear-friction method and a strut-and-tie model for joint behavior was shown in Section 11.7, and Example 11.5 presented the detailed design of a bracket for a precast concrete column. Additional design information pertaining to precast concrete connection design will be found in Refs. 18.9, 18.12, and 18.13.

FIGURE 18.31
Column-to-column connections.

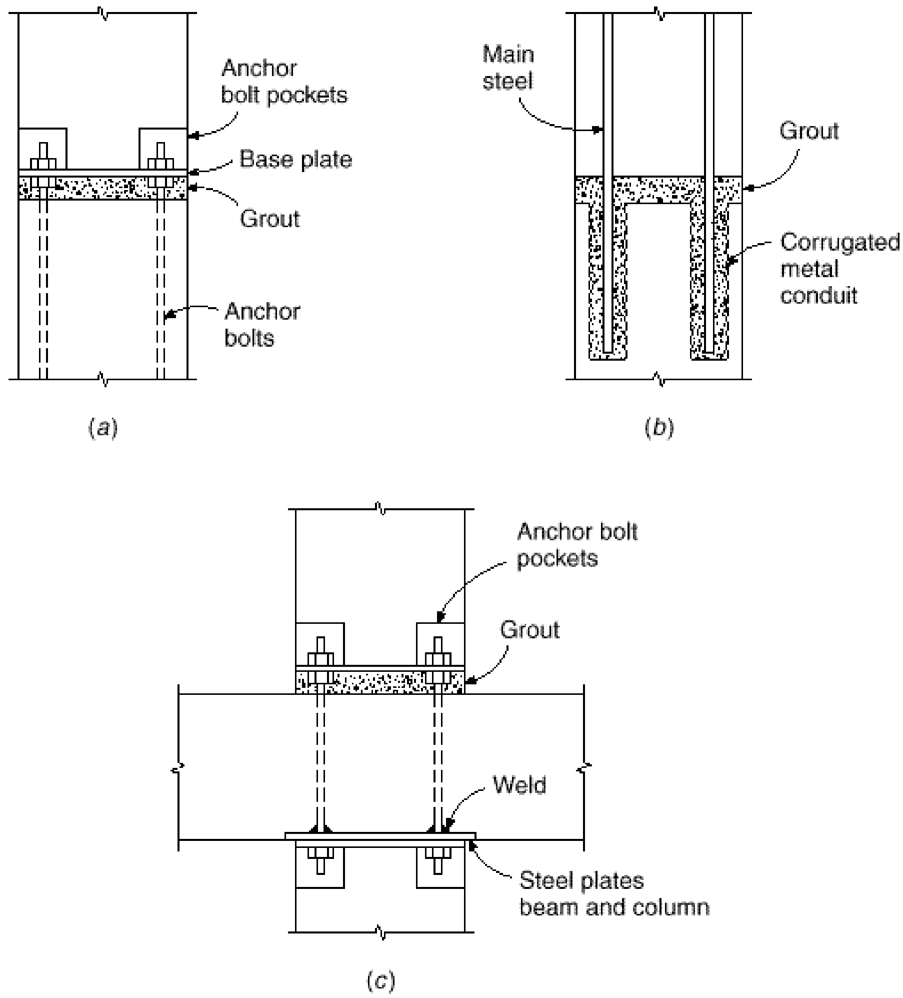
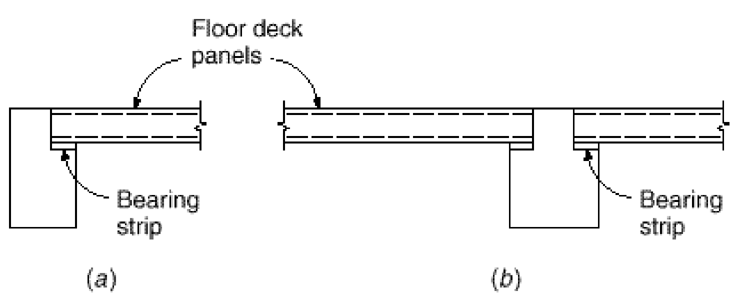


FIGURE 18.32
Slab-to-beam connections.



c. Structural Integrity

Precast concrete structures normally lack the joint continuity and high degree of redundancy characteristic of monolithic, cast-in-place reinforced concrete construction. Progressive collapse in the event of abnormal loading, in which the failure of one element leads to the collapse of another, then another, can produce catastrophic results.

For this reason, the structural integrity of precast concrete structures is specifically addressed in ACI Code 16.5. ACI Code 16.5.1 does not permit the use of "soft" connections that rely solely on friction caused by gravity forces. Full moment-resisting connections are unusual, but some positive means of connecting members to their supports, with due regard to the need to accommodate dimensional changes associated with creep, shrinkage, and temperature effects, is strongly recommended.

In addition, experience with precast structures has shown that the introduction of special reinforcement in the form of tension ties, though adding little to the cost of construction, can contribute greatly to maintaining structural integrity in the event of extraordinary loading, such as loads caused by extreme winds, earthquake, or explosion. This tension reinforcement is best arranged in a three-dimensional grid, usually on the column lines, tying the floors together vertically and in both horizontal directions. For precast concrete construction, ACI Code 7.13.3 and 16.5.1 require that tension ties must be provided in the transverse, longitudinal, and vertical directions of the structure and around its perimeter. Specific details vary widely. Although no specific guidance is offered in either the ACI Code or Commentary regarding steel placement or design forces, valuable suggestions will be found in Refs. 18.8, 18.9, and 18.13.

18.6 Engineering Drawings for Buildings

Design information is conveyed to the builder mainly by engineering drawings. Their preparation is therefore a matter of the utmost importance, and they should be carefully checked by the design engineer to ensure that concrete dimensions and reinforcement agree with the calculations.

Engineering drawings for buildings usually consist of a plan view of each floor showing overall dimensions and locating the main structural elements, cross-sectional views through typical members, and beam and slab schedules that give detailed information on the concrete dimensions and reinforcement in tabular form. Sectional views are usually drawn to a larger scale than the plan and serve to locate the steel and establish cutoff and bend points as well as to define the shape of the member. Usually a separate drawing is included that gives, in the form of schedules and cross sections, the details of columns and footings.

It is wise to include, on each drawing, the material strengths used for the design of the structure, as well as the service live load on which the calculations were based.

Typical engineering drawings will be found in Ref. 18.3.

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