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PRECAST/PRESTRESSED CONCRETE INSTITUTE

research fellowship

This paper presents a newly developed interior framing system for precast prestressed concrete multistory office buildings. The new system is optimized for gravity loads; however, it is also shown to have some features that would assist the structure in lateral load resistance. The system is composed of hollow-core slabs or double tees supported on 8 ft (2.40 m) wide inverted tee beams.

The beams are voided to improve structural and erection efficiency and to allow for possible use as utility housing. A beam thickness of 16 in. (400 mm) is shown to be adequate for a 40 x 36 ft (12.0 m x 11.0 m) bay. The corresponding conventional system would require a much thicker 2 ft (0.6 m) wide beam, thus increasing the structure height.

The beams of the proposed system are made continuous in the field by means of a cast-in-place top flange and continuity reinforcement. Thus, the seismic resistance, positive moment reinforcement, and deflection are considerably improved. The proposed system does not require column corbels which are expensive to produce, nor does it require field shoring which complicates construction.

A numerical design example of a typical interior span, including details of the beam as well as its connection with the column, is given. It is shown that the system meets all non-seismic design criteria of the ACI 318-89 Building Code; seismic design (Chapter 21) was not performed.

An economic analysis is conducted on a two-story 163 x 183 ft (49.7 x 55.8 m) office building. Costs are based on current prices in the Midwest region of the United States. Results of the analysis show a 13 percent savings over the conventional framing system. A spreadsheet computer program for structural design of the beam is available from the PCI for Apple Macintosh microcomputers.

The paper concludes with a brief description of an "all-dry" variation of the proposed system. It does not require cast-in-place concrete for structural integrity during erection, while maintaining the advantages of the proposed system.

Minimization of Floor Thickness in Precast Prestressed Concrete Multistory Buildings

Precast concrete is an economical and high quality product. Together with other benefits, such as short erection time, long span capability, aesthetic appearance and durability, precast concrete has been a competitive building material since it was first introduced in the United States. However, its penetration into the office building market has not been as successful as it has been in other sectors of the construction industry.

The main problem has been the inability of precast concrete to have a shallow framing system that is economical and yet provides for continuity and lateral load resistance. Large structural depth corresponds to increased building height which in turn increases the costs of architectural, mechanical and electrical systems, and increases lateral force effects on the structural system. It may prevent the opportunity of maximizing the amount of leasable space. For example, if the structural depth could be reduced by 2 ft (0.6 m) per floor, it would save 12 ft (3.6 m) of height for each six stories; in other words, allow a seven-story building of the same height as a six-story conventional building.

Simple (non-moment resistant) beam-to-column connections are predominantly used in precast building systems. This type of connection offers the advantages of ease and speed of construction but only at the expense of requiring shear walls or other lateral load resisting components. Also, simple spans deflect more and do not contribute to reduction in positive moment requirements. Finally, simple connections are often associated with column corbels which are expensive to produce. Because of these limitations, an improved system was developed in this study.

This paper begins with a summary of commonly used precast concrete building systems in the United States, Australia and Europe. Descriptions of the newly developed system and the construction sequence are presented next. Several methods of forming voids in the beam are addressed. A design example is given to demonstrate and to illustrate the design procedure involved.

A two-story office building is used as an example to demonstrate the economic feasibility of the proposed system. The possibility of housing utility ducts in the beam voids is mentioned. Two possible variations of the proposed system are described. Finally, the paper is summarized and a suggestion for future work is offered.

EXISTING SYSTEMS

Some of the commonly used precast building systems in the United States, Europe and Australia are described in this section. The American and Australian systems are covered in detail; only the main features of the European systems are given. More detailed discussion of the various systems is included in the full report (see Reference 1).

Conventional System[2]

At the present time, the most commonly used precast concrete office building framing system in the United States consists of hollow-core slabs or double tees, inverted tee beams and columns (Fig. 1). Precast prestressed or precast conventionally reinforced square columns are normally used. Corbels are usually provided to transfer beam gravity loads. Non-moment resisting connections are predominantly used in the conventional system.

Composite Dy-Core System[3]

The Composite Dy-Core System (Fig. 2) consists of precast Dy-Core hollow-core slabs, composite soffit beams and columns with cavities at beam level. The columns could be precast or cast-in-place (CIP). The Dy-Core slabs are produced by Finfrock Industries, Inc., Orlando, Florida. These slabs are typically 8 in. x 4 ft (2.4 x 1.2 m) and are made of high strength, normal weight concrete with strengths up to 8000 psi (55 MPa) when delivered to the site. Precast concrete soffit beams are typically 8 in. x 4 ft (2.4 x 1.2 m). The soffit

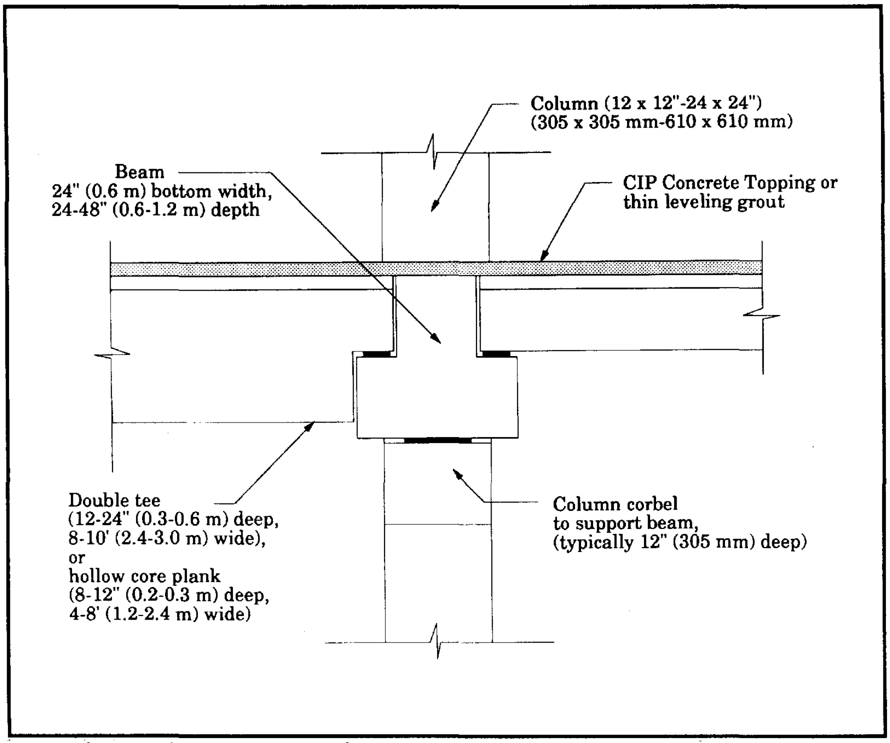

Fig. 1. Conventional system — Cross section through beam with typical dimension ranges.

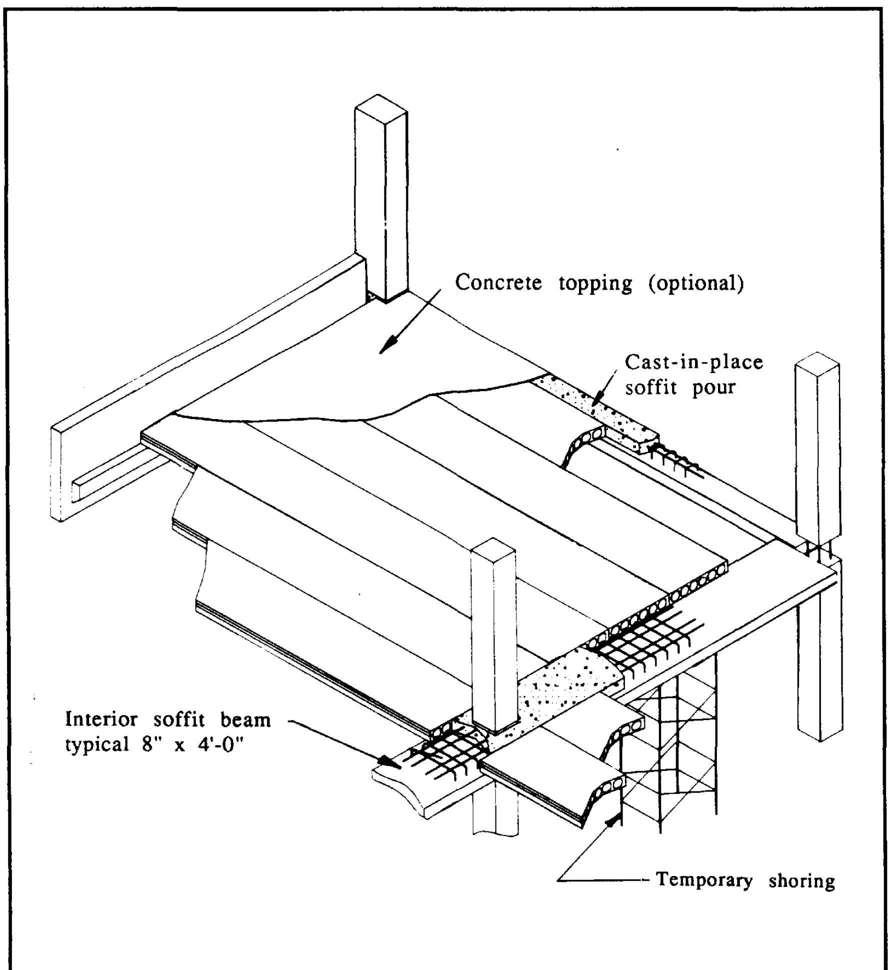

Fig. 2. Composite Dy-Core system.

beams are topped with CIP composite concrete.

Electrical and mechanical ducts may be placed within the cavities of the slabs and the CIP concrete beam section. Cavities in the precast columns at the beam level and temporary shoring are used to eliminate the need for column corbels. This also results in a composite moment-resisting connection. Temporary formwork is used for forming around the column area. This system is commonly used for office buildings with a 30 x 30 ft (9.1 x 9.1 m) typical bay size.

Floor construction can be staged in five steps, following column erection:

1. Precast soffit beams are placed on shoring between columns.
2. Precast Dy-Core slabs are set on the soffit beams to provide a platform for workers to proceed with construction. At this stage, the shear keys between slabs are grouted to create a rigid diaphragm.
3. Top beam continuity reinforcing steel is placed between as well as outside of the column reinforcement. Bulkheads are placed in the Dy-Core voids to block the flow of the CIP concrete into them.
4. The CIP concrete is poured to form the top portion of the soffit beams. If the 2 in. (51 mm) concrete topping is not used, a thin self-leveling noncomposite topping material is used to prepare the deck for carpeting or another floor covering.
5. Finally, the soffit shoring is removed when the composite pour reaches the required strength.

MSF Wideslab System[4]

The MSF Wideslab System is a composite system. The system was first used in 1968 by the Mid-State Filigree Systems, Inc., Cranbury, New Jersey. The main components of the system are 2 to 3 in. (51 to 76 mm) thick precast prestressed concrete slabs which are reinforced with steel trusses and function both as forms and as part of the complete slab (see Fig. 3). Three types of Wideslabs are produced: one-way and two-way conventionally reinforced, and one-way prestressed.

The bottom steel in the transverse

Fig. 3. Filigree Wideslab.

direction in two-way slabs is spliced with a patented coupling system utilizing field-installed splice bars to provide the continuity. Wideslab units are constructed in widths of 8 ft (2.4 m) and lengths up to 60 ft (18.3 m). If weight reduction of the slabs is desired, voids could be formed at the site. The minimum finished thickness to form voids using the Filigree void forms is 6 in. (152 mm). Temporary shoring is needed during erection.

In the construction process, the Filigree Wideslab units are first placed on temporary falsework or props. Then conduits, pipe, wire chases and other subfloor utilities are installed. Top reinforcing steel is placed as required. The top concrete is then poured to form the finished composite deck. Temporary falsework is removed as soon as the CIP concrete achieves adequate strength. Fig. 4 illustrates one of the construction details of the system. This type of construction provides a nearly monolithic system, which has significant wind and seismic resistance.

PSI System[5]

Another composite system, the PSI System, is currently being produced by Prestressed Systems, Inc., Miami, Florida. The system consists of precast prestressed joists and soffit beams, and conventionally reinforced CIP slabs. The joists are similar in shape to a web of a double tee (Fig. 5a). Joist depth ranges from 8 to 28 in. (200 to 710 mm). Joists built-up with CIP concrete topping are used for cases where a larger depth is needed.

The spacing of joists varies from 3 ft 6 in. to 8 ft 8 in. (1.1 to 2.6 m) for spans of 24 to 80 ft (7.3 to 7.9 m). The soffit beams, on the other hand, are heavily pretensioned 24 x 6½ in. or 16 x 9½ in. (610 x 170 mm or 410 x 240 mm) sections that furnish positive reinforcement and stirrups (see Fig. 5b). The beam centerline to centerline is typically 24 to 26 ft (7.3 to 7.9 m) for office buildings. Slabs [3 in. (75 mm) thick] are used for joists up to 16 in. (410 mm) deep, and 4 in. (100 mm) slabs are normally used for deeper joists. Formwork and shoring under the soffit beams are required during construction.

Quickfloor System[6]

The Quickfloor System was developed in Australia (see Fig. 6). The precast concrete beam section is similar to the hollow-core slab section it supports except that a part of the top flange is blocked out during precasting, which results in a section with webs and bottom flange only. Beam thickness is typically 11 to 12 in. (275 to 300 mm), and width is in the range of 4 to 6 ft (1200 to 1800 mm). The precast concrete floor units, on the other hand, are typically 16 in. (400 mm) thick. Thickness of beams, floor units, and a 3 in. (70 mm) topping result in a total depth of 30 in. (770 mm) for a 60 x 30 ft (17.8 x 9.0 m) bay. The columns are one-story elements.

Construction of this system is similar to the Composite Dy-Core System. First, the columns are set up. Normally, four temporary props are placed between column centerlines: near the columns and at third points of the beam span. Next, the beams are set on these temporary props. Floor units, which usually require 4 to 5 in. (100 to 125 mm) of bearing width, are placed on the edges of these beams. Addition-

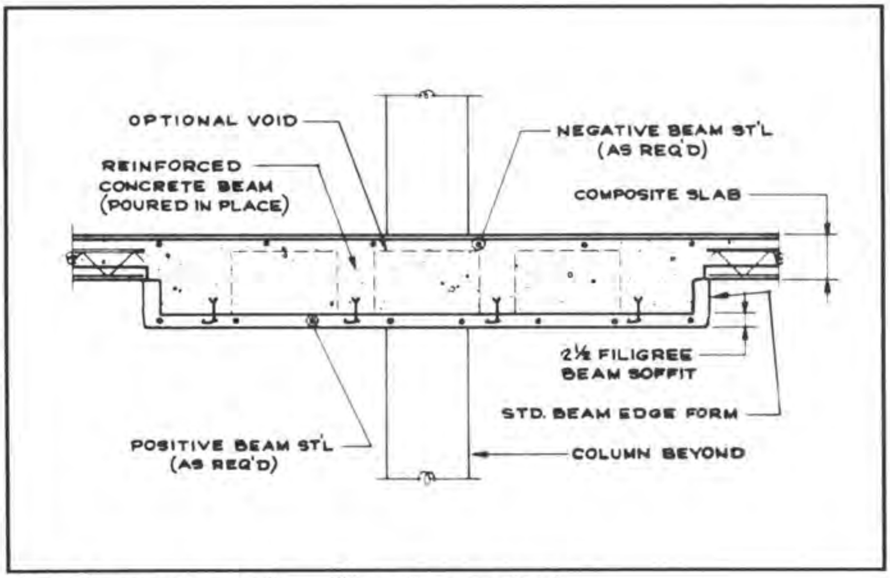

Fig. 4. Construction details — Filigree beam and slab.

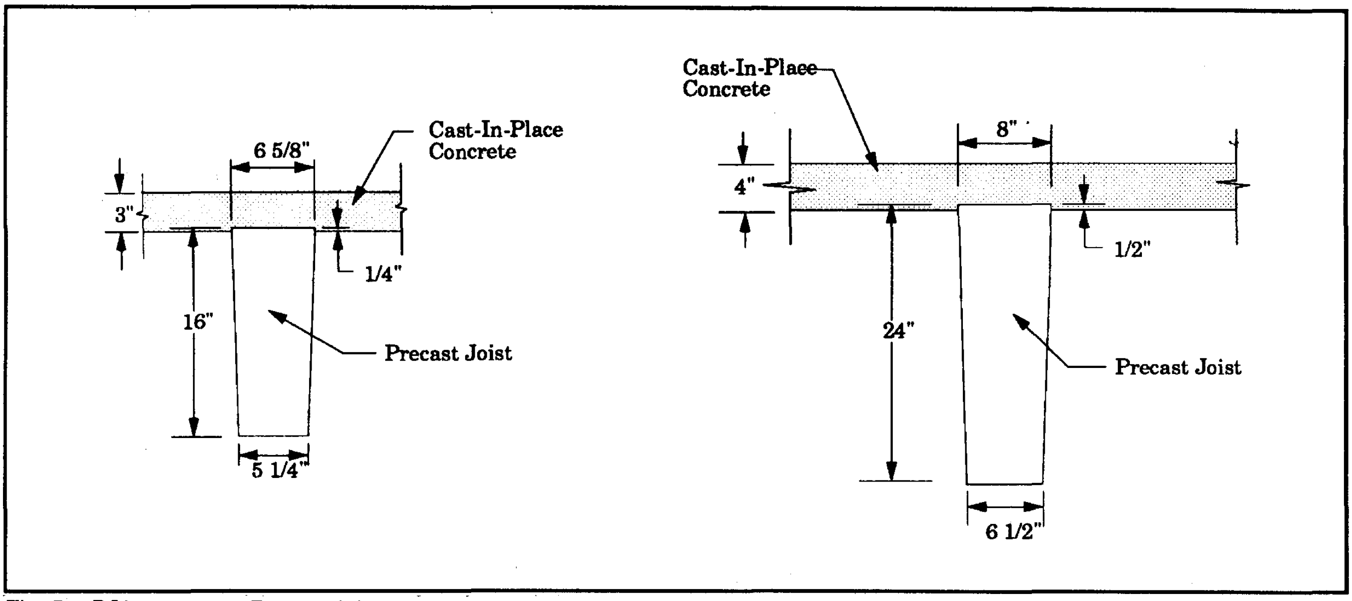

Fig. 5a. PSI system — Precast joists.

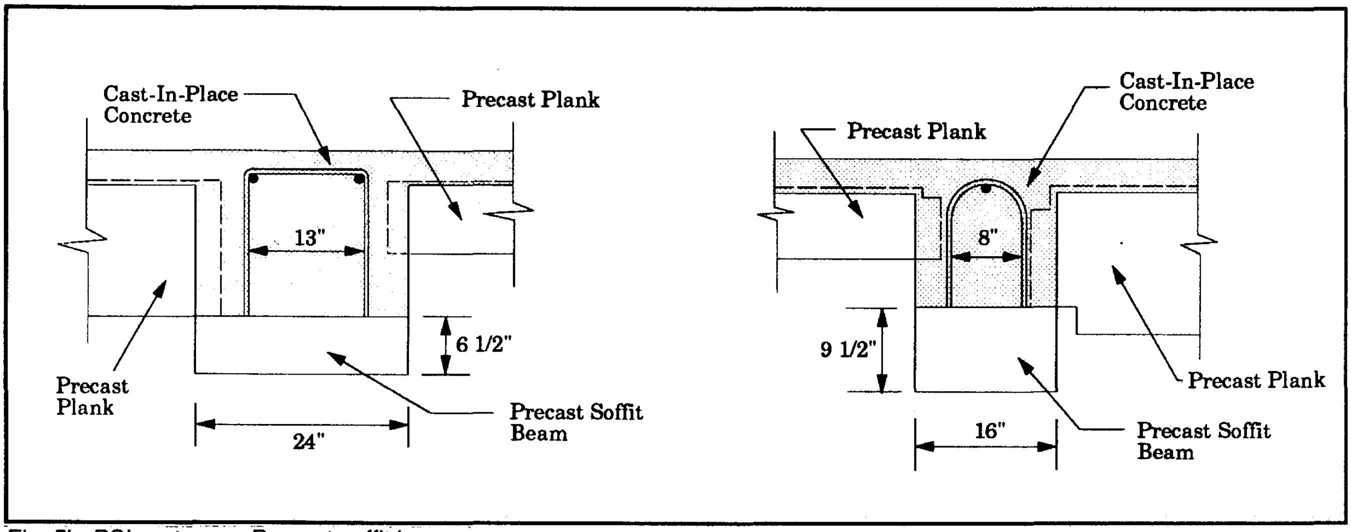

Fig. 5b. PSI system — Precast soffit beams.

al reinforcing steel for the negative moment, shear and torsion is placed in the beam, and the entire floor area is covered with a reinforcement mesh to control cracking. Finally, CIP concrete is poured.

The beams are produced in a way similar to the hollow-core slabs, i.e., by means of a slipform or extrusion process. This is the main attractiveness of the system since no special equipment is required and automation reduces plant labor. The system is now being marketed in the United States.

Other Systems

The IMS System[7] is a precast post-tensioned system which was first used in Yugoslavia. The system utilizes two-way precast concrete floor panels in which both the top and bottom of the unit are closed, similar to a waffle slab with a bottom flange. This system relies on the post-tensioning tendons being placed at the column lines and at third points between columns to support the floors. No column corbels or beams are used (see Fig. 7).

The BVM-TIP System[8] is a complete precast concrete system with standardized connections. One of the two types of connections used is described here. The system utilizes a knife-edge steel plate which protrudes from the column to support the floor system, as shown in Fig. 8. A welded angle and box assembly attached to the column is used to support the hollow-core slabs at the column area.

PROPOSED SYSTEM

The proposed system is illustrated in Figs. 9, 10 and 11. It consists of multistory precast concrete columns, 8 ft (2.4 m) wide inverted tee beams, and standard hollow-core slabs. A full concrete block-out (cavity) is made in the column at each beam level during precasting while allowing the column reinforcement to run continuously through the entire length. The cavity is slightly deeper than the beam thickness. It is filled with CIP concrete during construction. The thickness of the

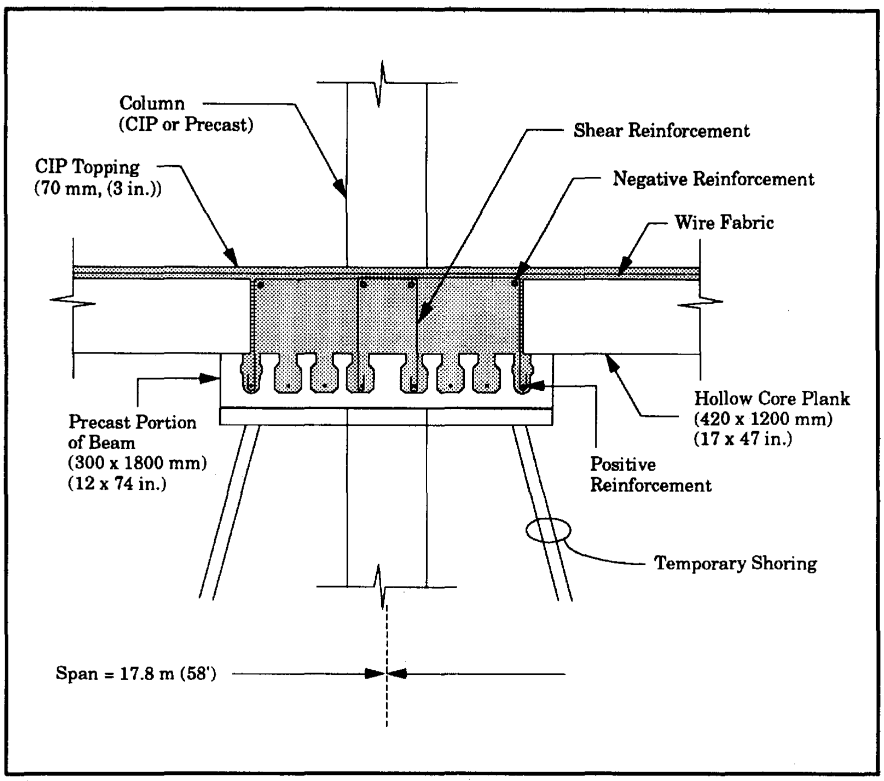

Fig. 6. Quickfloor system.

beam is dependent upon the bay size and loading. Voids are created within the beam to reduce self weight and improve its structural performance.

Beam continuity is desired to reduce its deflection and create moment distribution between the positive and negative areas. The continuity is achieved using CIP concrete and field placed reinforcement in the negative moment zone. A steel cage is transversely inserted in the CIP joint through the column cavity in order to ensure resistance of the joint in that direction.

Nonprestressed strands may be used as an alternate to conventional reinforcing bars to resist negative moment induced by the continuity of the beam, thereby achieving a low material-to-labor cost ratio. The top reinforcing steel is staggered as permitted by the ACI 318-89 Building Code[9] for placement of reinforcement.

To allow room for placement of the negative reinforcement in the field, a portion of the top flange of the beam is blocked out in the plant and field cast after steel placement. Determination of the length of the field cast flange is based on the development length requirements of the negative reinforcement.

Steel angles are attached to the column to serve as temporary beam supports during construction. Two of the many features of the proposed system are that no field welding or shoring is required. Fig. 10 shows that the outside webs of the beam are deeper than the middle web. This is to allow for placement of a platform for field work.

It must be emphasized that the proposed system is applicable to inverted

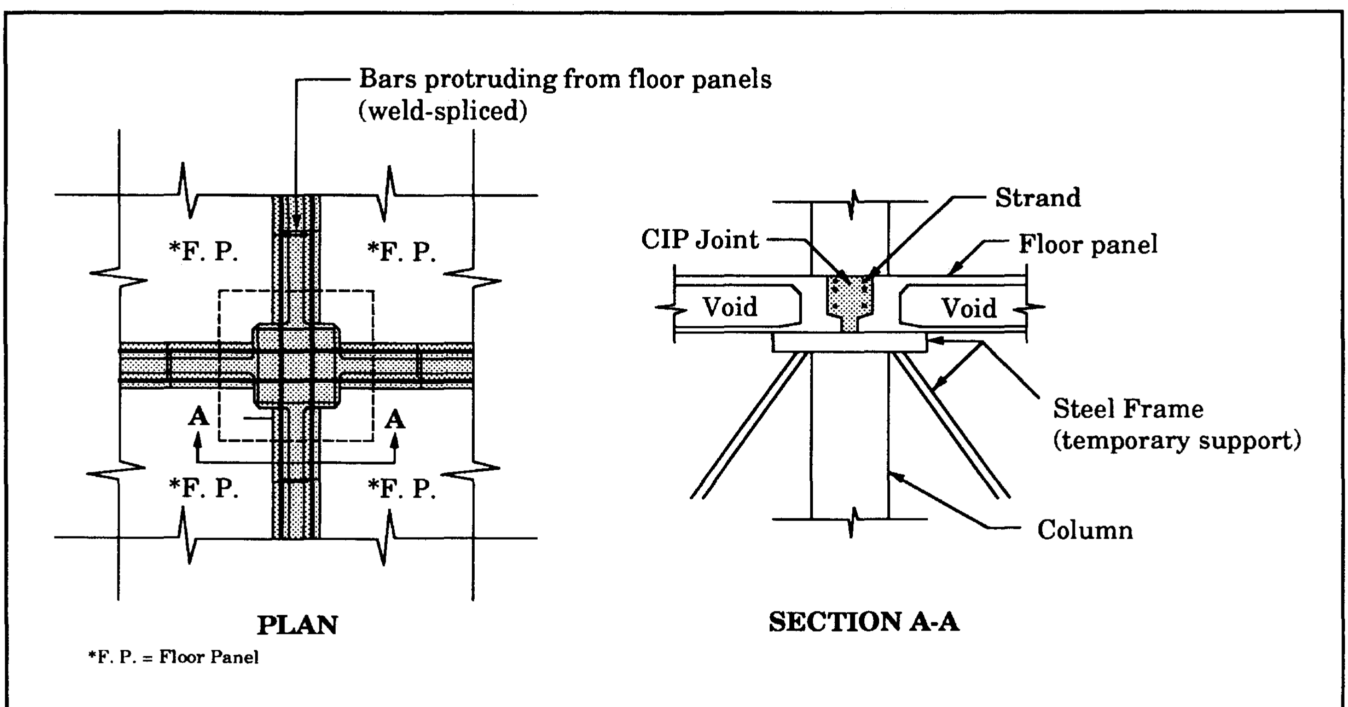

Fig. 7. IMS system.

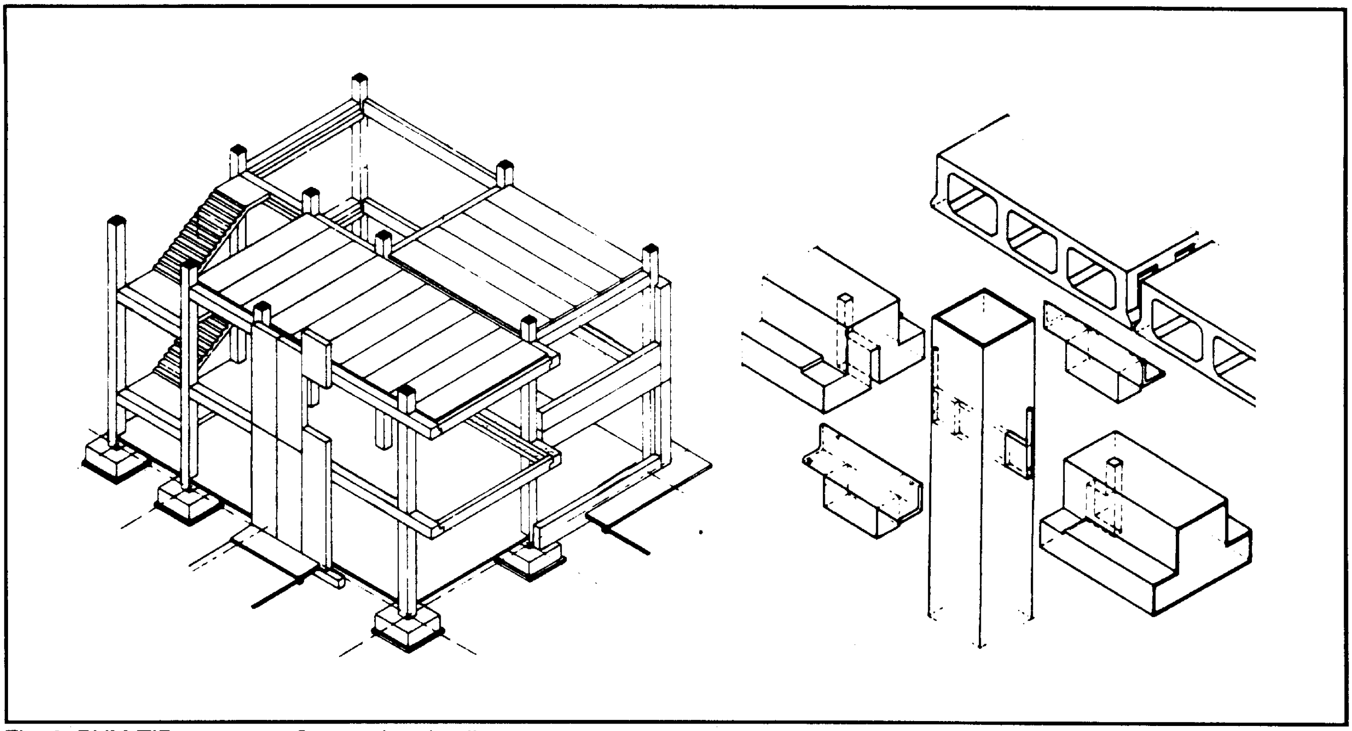

Fig. 8. BVM-TIP system — Connection details.

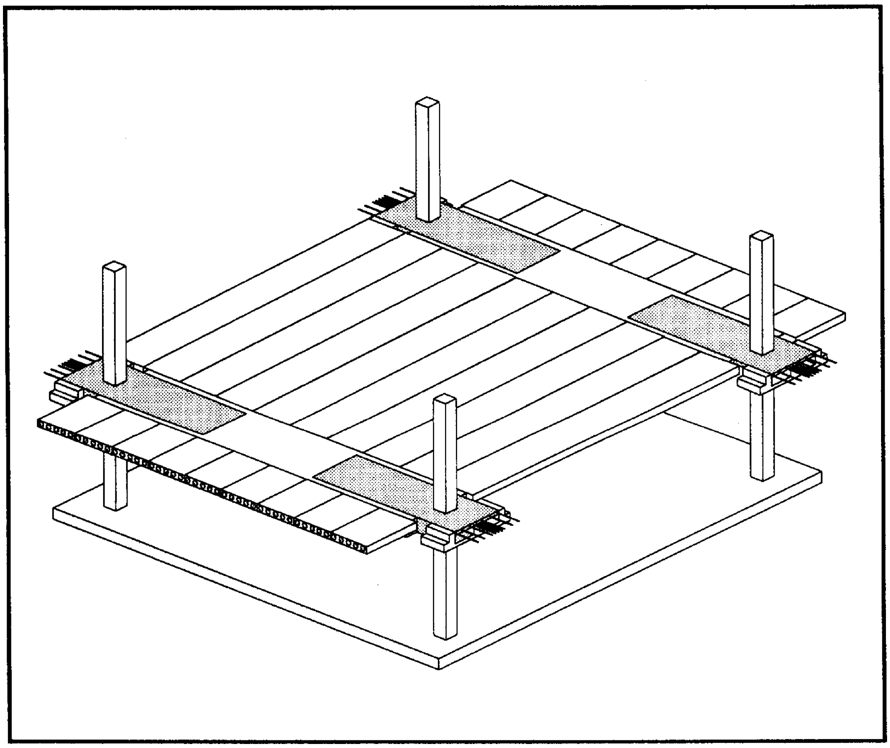

Fig. 9. Proposed system.

tee beams of widths other than 8 ft (2.4 m). The selection of a particular beam width should be determined on the basis of overall system economy, not that of the beam alone, and of satisfactory structural performance. For example, a wide beam could reduce the hollow-core slab depth and cost. Also, the wider the beam, the shallower it becomes, thus reducing the overall building height and associated costs. On the other hand, an extremely wide beam could be torsionally unstable, especially during construction. The authors believe that an 8 ft (2.4 m) width is reasonable.

Construction Sequence

The construction sequence of the proposed system is shown in Fig. 12 and described here.

Stage I — Columns are set up and temporarily braced. Angles are attached to the column using threaded rods which run through sleeves in the column, to serve as temporary supports. Formwork for the CIP joint is placed on the angles.

Stage II — Beams are placed on temporary supports.

Stage III — Steel cages are inserted transversely through the columns.

Stage IV — Negative moment reinforcement is placed. The bar spacing should be smaller in the vicinity of the column than away from it. The bars, or nonprestressed strands, are extended to the edge of the precast top flange to meet the development length requirements.

Stage V — Concrete for the top flanges, column voids and transverse joints is poured. Plugs may be needed to prevent concrete from going too far into beam voids. A platform could be placed on top of the end webs to provide a working area for concrete finishing.

Stage VI — As soon as the CIP concrete has achieved adequate strength, the temporary supports and

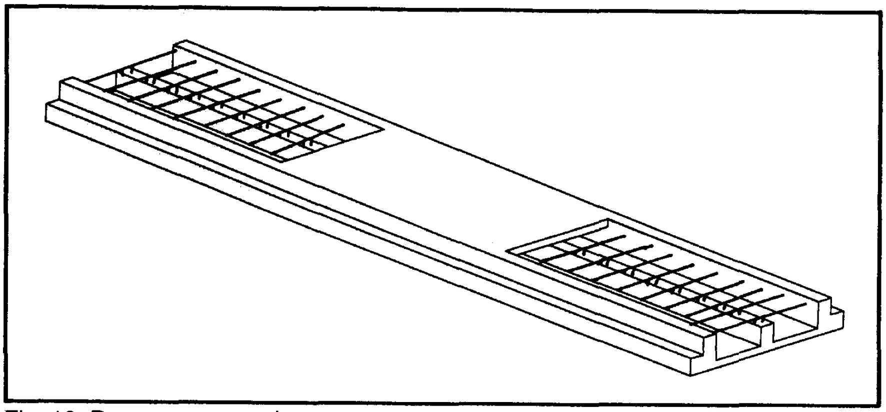

Fig. 10. Precast concrete beam.

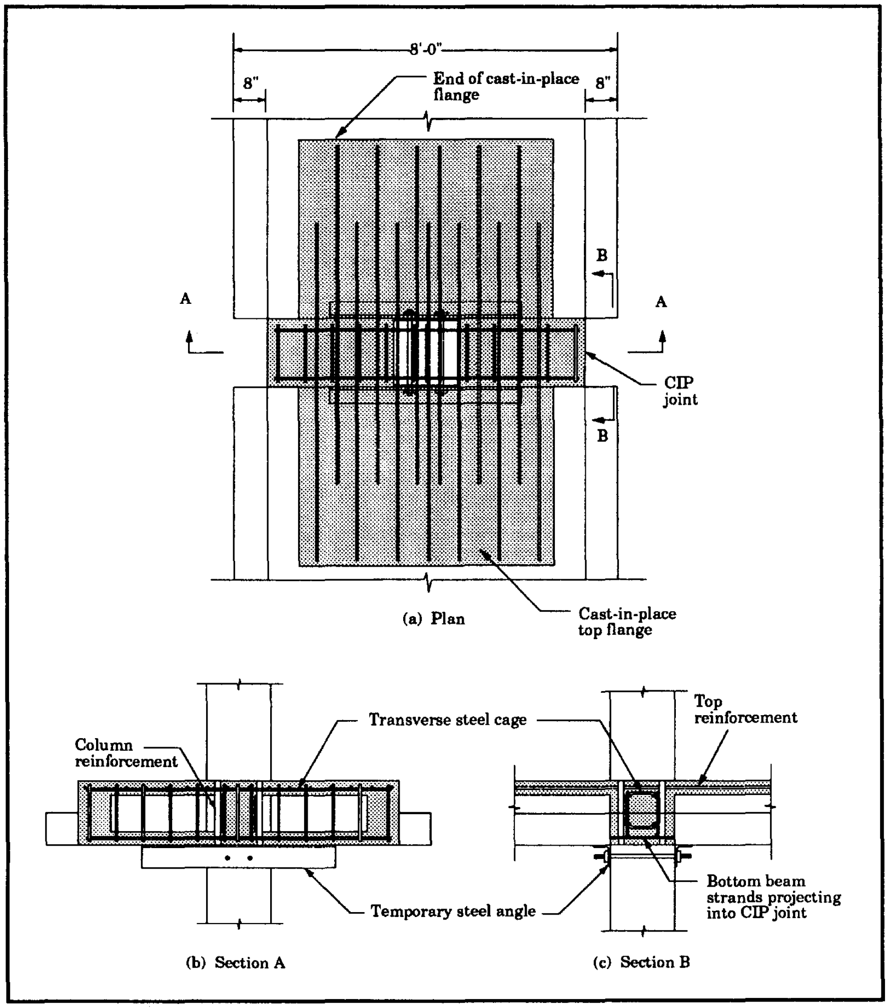

Fig. 11. Details of system in beam-to-column area.

formwork are removed. This stage may also be proceeded during or after Stage VII.

Stage VII — The hollow-core slabs are placed. There is no limitation to the order in which the contractor places the slabs; the system can be easily designed for a worst case scenario. An insert system can be used for the connection between the slabs and the beam to improve diaphragm action and structural continuity in the hollow-core direction. This would be done by embedding wire inserts in the beam side and connecting them with threaded rods placed in the joints between hollow-core slabs. The joints would then be grouted. Other useful details are available in Reference 10, which deals with the seismic resistance of floors utilizing untopped hollow-core slabs. The final step is to place CIP topping, if required, or a thin leveling grout to prepare for carpeting.

BEAM VOIDS

Void-forming cost may be higher than the cost of the concrete saved. However, a voided beam is better for the overall system. Benefits of a voided beam include improved structural efficiency, reduced beam weight and the corresponding crane capacity, and reduced load on the supporting columns and foundation.

Voids could be formed using one of the following methods (see Fig. 13). Production economy will dictate the method adopted by the producer.

1. In-place cardboard boxes or Sonovoid Fibre Tubes.[11]
2. Removable inflatable tubes, similar to those used for Flexicore slabs.
3. Encased aggregates, similar to the material used for Span-Deck slabs. (Plastic or special paper may be used to contain the aggregates and control the void shape.)
4. Expanded polystyrene insulation boards.
5. Retractable steel void forms.

DESIGN EXAMPLE

Given the interior bay of a multistory, multibay office building whose dimensions are shown in Fig. 14, it is required to design the beam of the proposed system and its connection with the columns.

The floor is subjected to a superimposed dead load of 35 psf (1.7 kPa), live load of 50 psf (2.4 kPa), and construction load of 20 psf (1.0 kPa). The materials assumed to be available are normal weight concrete [unit weight = 150 pcf (2400 kg/m^3)] whose strength is 5000 psi (34.5 MPa) at service and 3500 psi (24.1 MPa) at prestress release, 270 ksi (1860 MPa) low-relaxation seven-wire strands, Grade 60 reinforcing bars, A36 structural steel shapes, and threaded rods. Also, the structure is assumed to be subjected to average environmental conditions for time-dependent volume change analysis.

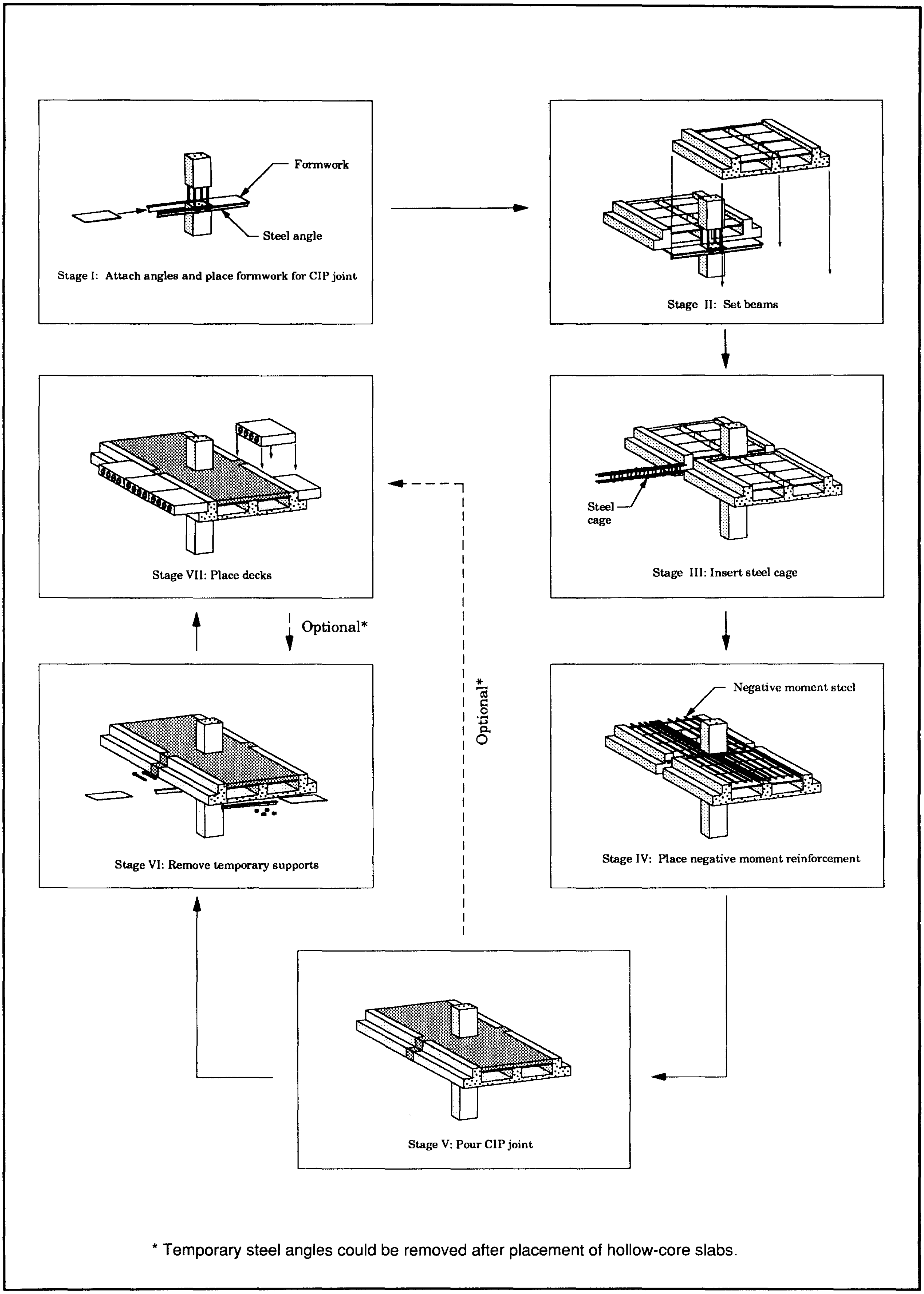

Fig. 12. Construction sequence.

A preliminary framing system is shown in Fig. 14. It consists of 8 in. (200 mm) hollow-core slabs, weighing 61 psf (2.9 kPa), supported on an 8 ft (2.4 m) wide, 16 in. (410 mm) thick hollow beam. The depth of the hollow-core slabs was selected from the PCI Design Handbook.[2] A deeper section may be required depending on the licensed slab system employed.

Sections A, B, C and D of the beam are shown in Fig. 15. The CIP concrete in the top flange, Section A, and in the beam-to-column connection, Sections C and D, render the beam continuous. All interior columns are assumed to be 16 x 16 in. (410 x 410 mm) precast concrete multistory columns. Voids at the ends of the beam are filled with CIP concrete, for a length of 2 ft (0.6 m) from the face of support, to improve the strength of the beam-to-column connection.

The weight of the precast concrete beam is about 27.4 kips (12,400 kg), which is equivalent to 103 ft (31.4 m) of a 16 x 16 in. (410 x 410 mm) normal weight concrete column, or 46 ft (14.0 m) of a 24 x 24 in. (610 x 610 mm) column. The crane capacity may be controlled by the beam or column weight depending on the column size and length.

For brevity, only highlights of the beam design are given here. Details of the design are covered in the full report.[1] Also, a computer program is available from the PCI at nominal charge to aid in the design of the beam. The program was written on the Apple Macintosh as a Wingz spreadsheet application. It can be easily adapted to IBM PC software such as LOTUS 1-2-3.

Step 1. Preliminary Flexural Design

(a) Strength[12]

The critical sections in strength design calculations are at midspan and at column face. In lieu of a more rigorous method, such as the Equivalent Frame Method in Chapter 13, ACI 318-89 Code,[9] the approximate coefficients of Section 8.3.3 are assumed to be valid.

Therefore, the maximum positive factored load moment at midspan is equal to 9965 in.-kips (1130 kN·m).

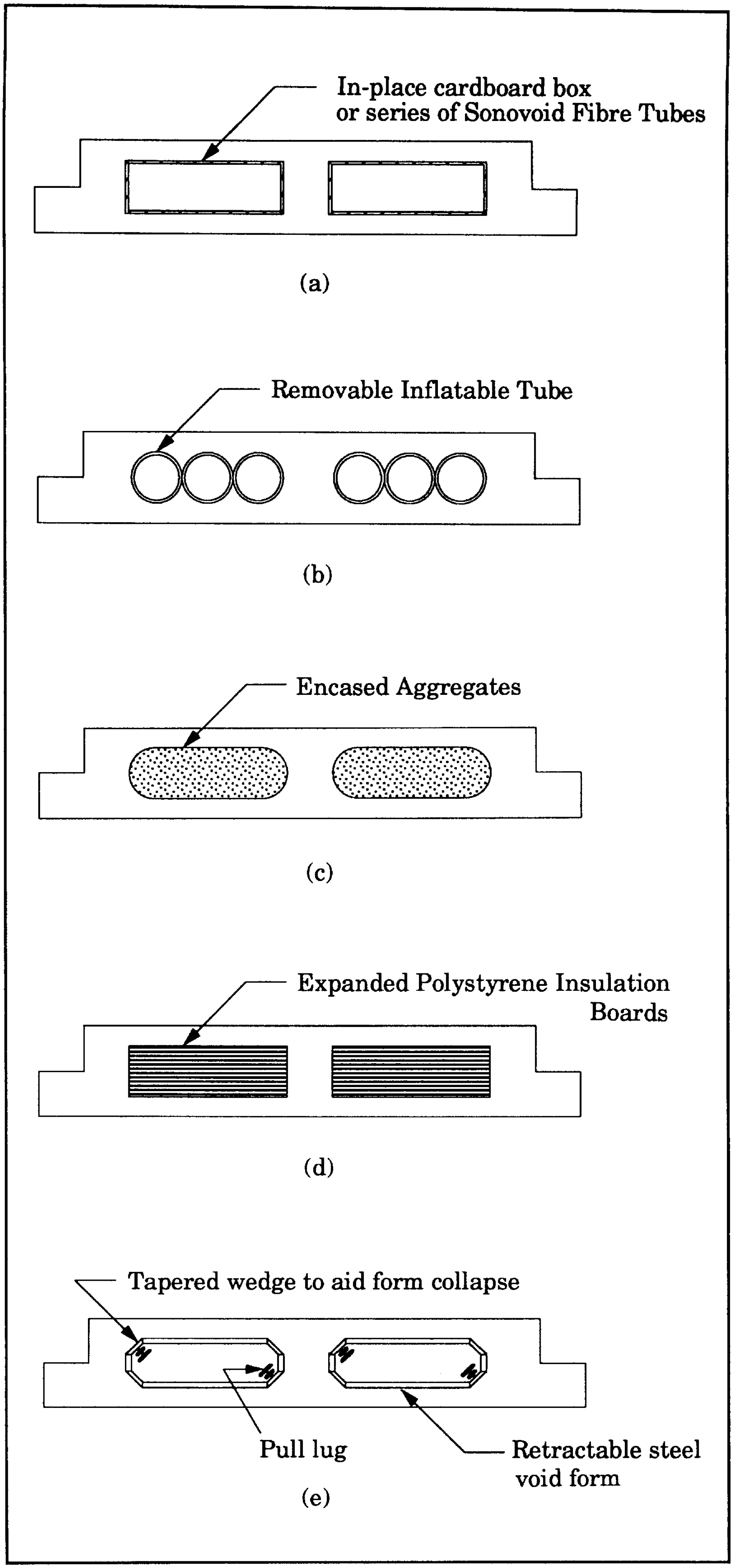

Fig. 13. Possible methods of forming beam voids.

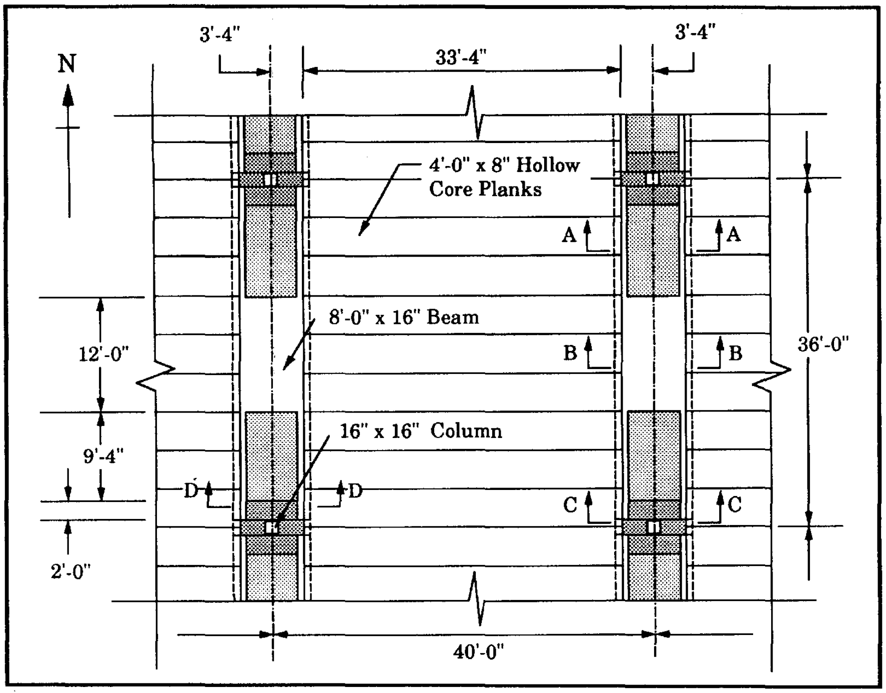

Fig. 14. Building layout — Plan.

The self weight of the beam is considered to be applied to a simple span and the remaining loads to a continuous span.

The corresponding negative moment is equal to 10,773 in.-kips (1220 kN·m).

Assuming steel stress at ultimate flexure equal to 255 ksi (1760 MPa) and an effective depth of 14.25 in. (360 mm), the approximate positive and negative steel areas are 3.38 in.^{2} (2180 mm^2) [or 24 - ½ in. (13 mm) diameter strands] and 3.66 in.^{2} (2360 mm^2) [or 26 strands].

Note that the positive moment strands are prestressed whereas the negative moment strands are not prestressed.

(b) Working Stress[2]

The minimum amount of prestress force is determined such that the maximum stress at midspan does not exceed $6\sqrt{f'_c}$, according to the ACI 318-89 Code.[9]

Using a service load moment equal to 6730 in.-kips (760 kN·m), the solution for effective prestress, P, yields $P \geq 544$ kips (2420 kN).

With 24 strands provided, the effective stress becomes 148 ksi (1020 MPa), which is at an acceptable level.

If a time-dependent loss of 15 percent is assumed, the initial prestress force must be greater or equal to 640 kips (2850 kN).

Step 2. Detailed Flexural Design

(a) Strength

The iterative strain compatibility procedure by Skogman et al. is followed to determine the flexural strength of the beam.[13] It is particularly suitable since strands in the negative moment zone are not prestressed and, thus, the ACI Code Eq. (18-3) would not be applicable. With the steel provided, the nominal strength at the positive moment section is:

$$\phi M_n = 10{,}846 \text{ in.-kips (1230 kN·m)} > 9965 \text{ in.-kips (1130 kN·m)} \quad \text{ok}$$

and at the negative moment section the nominal strength is:

$$\phi M_n = 10{,}958 \text{ in.-kips (1240 kN·m)} > 10{,}773 \text{ in.-kips (1220 kN·m)} \quad \text{ok}$$

(b) Development of Negative Moment Steel

According to the PCI Design Handbook[2] (p. 4-20), the minimum development length is:

$$L_{d,\min} = \left(f_{ps} - \frac{2}{3} f_{se}\right) d_b$$

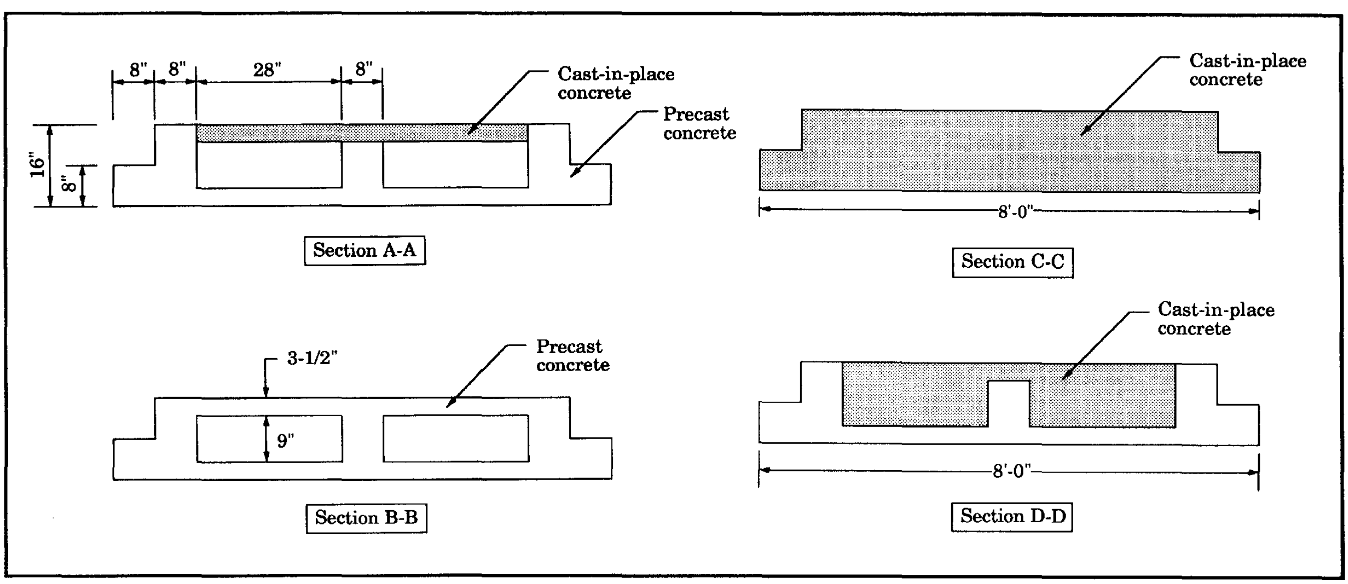

Fig. 15. Building layout — Section details.

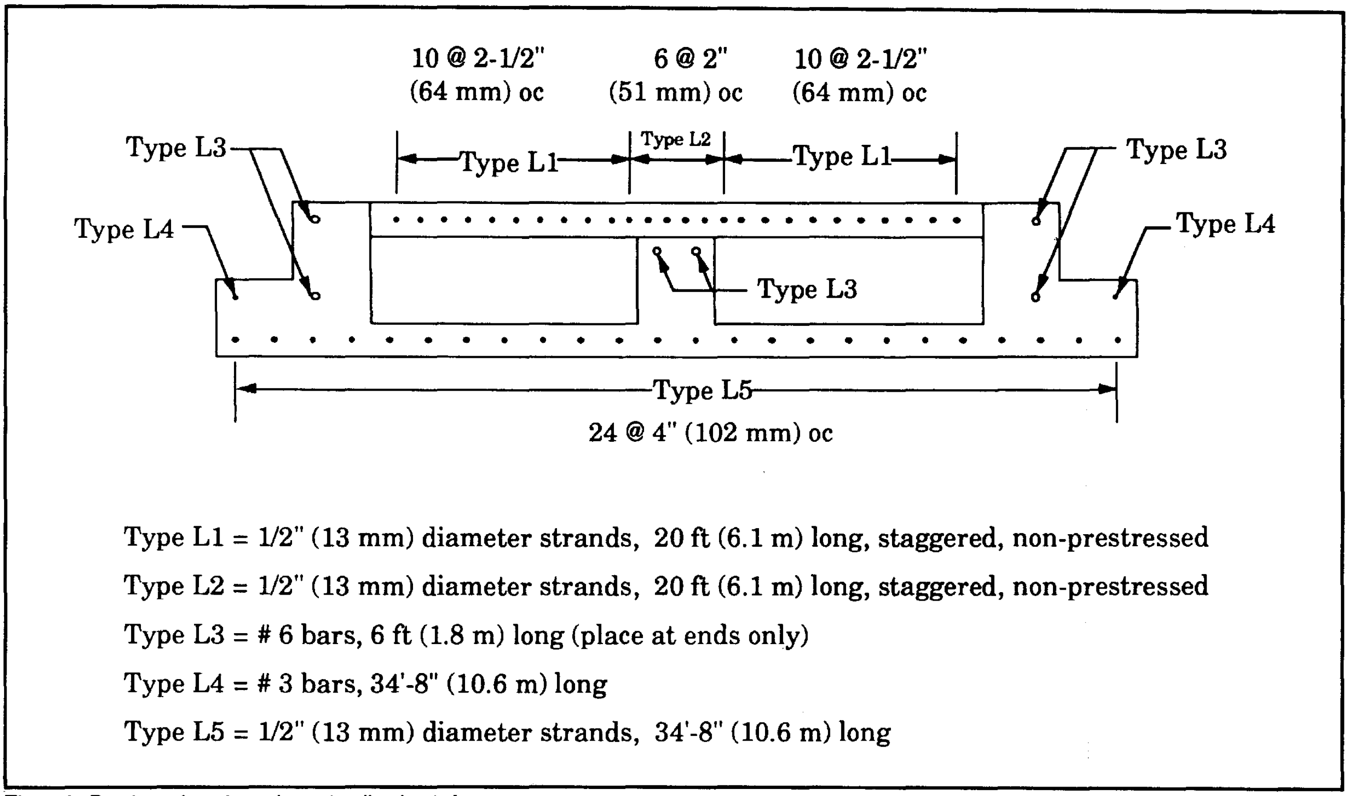

Fig. 16. Design sketch — Longitudinal reinforcement.

where f_{ps} is the steel stress at ultimate flexure, found here to be equal to 241.1 ksi (1660 MPa), and f_{se} is the effective prestress, assumed equal to -25 ksi (-172 MPa) to account for creep and shrinkage only, since the steel is not pretensioned.

The corresponding $L_{d,min.}$ = 127 in. (3.2 m). Therefore, the outside 11 ft 4 in. (3.5 m) of the top flange are CIP and the middle 12 ft (3.7 m) is precast.

(c) Working Stress[2]

Section 1 is at a transfer length distance from the beam end, i.e., 50 d_b = 50 (0.5) = 25 in. (640 mm). Sections 2 and 3 are at the changeover location from CIP to precast top flange, and Section 4 is at midspan.

Table 1 summarizes the stresses at prestress release and at service. The only stress exceeding the Code limits

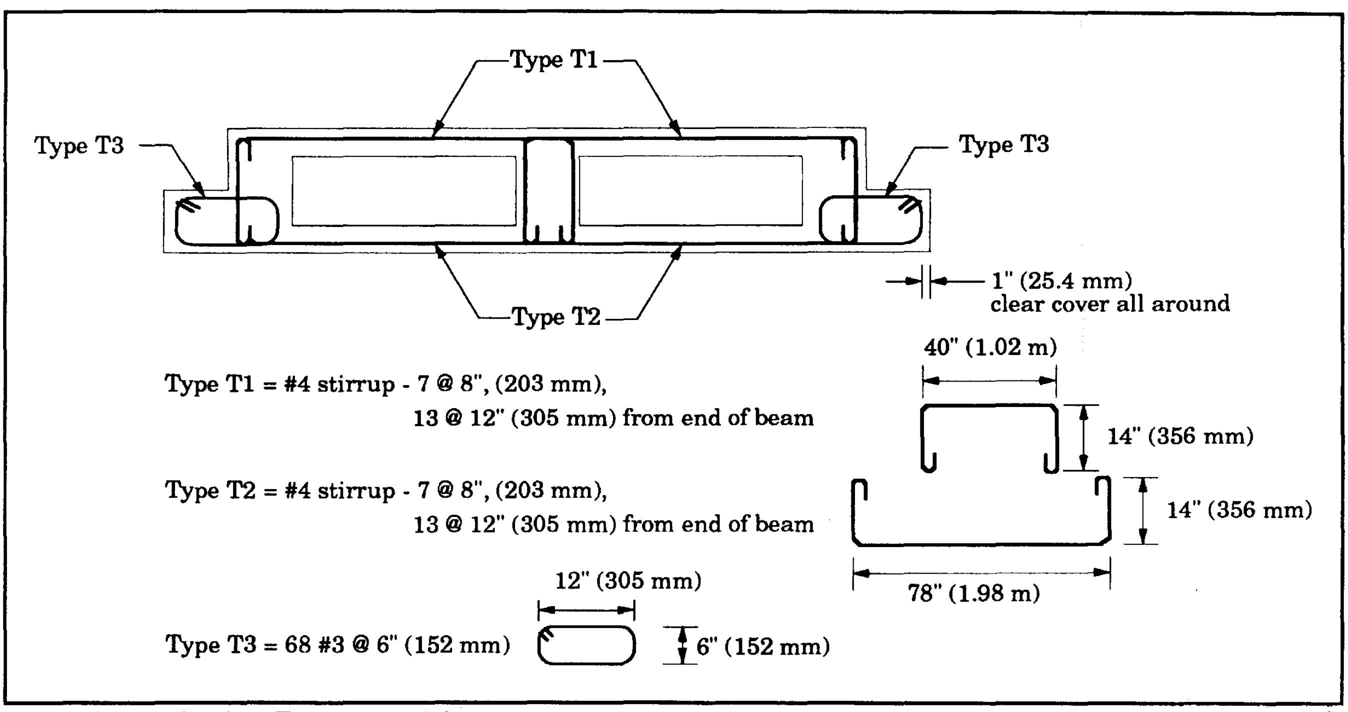

Fig. 17. Design sketch — Transverse reinforcement.

Table 1. Working stresses at critical sections (ksi).

	Section 1 at release		Section 2 at release		Section 3 at release		Section 4 at release		Section 4 at service	
Load	f_b	f_t	f_b	f_t	f_b	f_t	f_b	f_t	f_b	f_t
$\frac{P}{A}$	-0.941	-0.941	-0.941	-0.941	-0.708	-0.708	-0.708	-0.708	-0.602	-0.602
$\frac{Pe}{Z}$	-0.931	+1.938	-0.931	+1.938	-1.026	+1.182	-1.026	+1.182	-0.872	1.005
$\frac{M_d}{Z}$	+0.137	-0.285	+0.558	-1.162	+0.372	-0.429	+0.427	-0.492	0.517	-0.595
$\frac{M_{sd}}{Z^c}$									0.873	-1.006
$\frac{M_L}{Z^c}$									0.509	-0.586
Stresses	-1.735	+0.712	-1.314	-0.165	-1.362	+0.045	-1.307	-0.018	+0.424	-1.784
Allowable	$-0.60f'_{ci}$	$6\sqrt{f'_{ci}}$	$-0.60f'_{ci}$	$-0.60f'_{ci}$	$-0.60f'_{ci}$	$3\sqrt{f'_{ci}}$	$-0.60f'_{ci}$	$-0.60f'_{ci}$	$6\sqrt{f'_c}$	$-0.45f'_{ci}$
stresses	-2.100	+0.355	-2.100	-2.100	-2.100	+0.177	-2.100	-2.100	+0.424	-2.250
	ok	High*	ok	ok	ok	ok	ok	ok	ok	ok

Metric (SI) conversion factor: 1 ksi = 6.895 MPa.
* See Step 2, Section c for discussion.

is the tensile stress at release at the top of Section 1. According to the PCI Design Handbook[2] (p. 2-46), this stress is less than 800 psi (5.5 MPa) and is permissible if an amount of bonded steel is provided to resist the entire tension force. The calculation yields an area equal to 0.92 in.^{2} (590 mm^2). This area will be added to the longitudinal reinforcement required to resist torsion.

(d) Crack Control of Negative Moment Section

Since the negative moment area is not prestressed, a crack control analysis is required. Using Section 10.6.4 of the ACI Code,[9] the quantity z can be shown to be equal to 71.4 kips/in. (12.5 kN/mm), which is less than the maximum for both interior exposure [175 kips/in. (30.6 kN/mm)] and exterior exposure [145 kips/in. (25.4 kN/mm)].

Step 3. Combined Shear and Torsion[14,15]

(a) Maximum Torsion and Corresponding Shear

The first critical load combination to consider is when hollow-core slabs and full construction loads are introduced to one side only of the beam, i.e., the condition of maximum torsion and the corresponding shear. This case produces V_u = 55.1 kips (245 kN) and T_u = 1416 in.-kips (160 kN·m), at a section h/2 [= 8 in. (203 mm)] from the support face.

A detailed analysis shows that voided beam section shape selected has a superior performance to a solid section.[1] The amount of torsion reinforcement needed is A_t = 0.113 in.^{2}/ft (240 mm^2/m), and the amount of shear reinforcement needed is A_v = 0.208 in.^{2}/ft (440 mm^2/m). These two quantities will be combined with the transverse flexural reinforcement to determine the total tie requirements.

The area of longitudinal torsion reinforcement needed is 1.71 in.^{2}/ft (3620 mm^2/m), which must be added to the 0.92 in.^{2} (590 mm^2) required to control release stress (see second paragraph in Step 2, Section c) to produce 1.71 + 0.92 = 2.63 in.^{2} (1700 mm^2). Six #6 bars spread around the perimeter would be adequate.

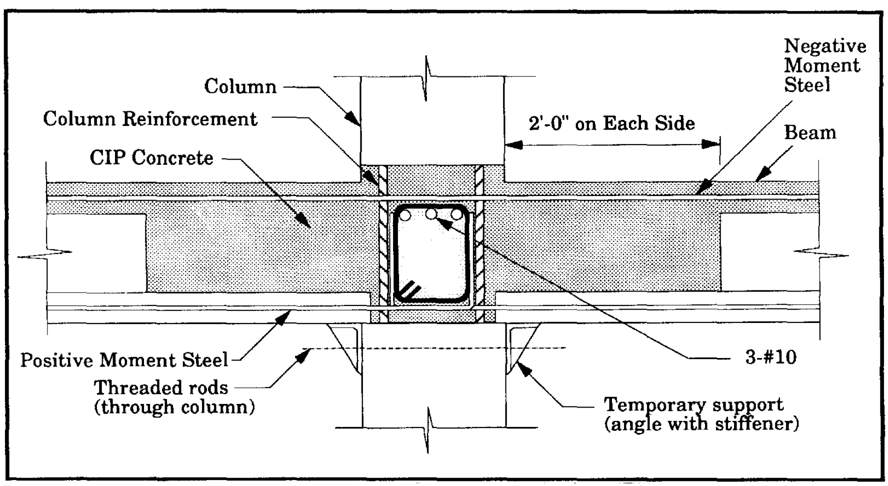

Fig. 18. Beam-to-column connection — Section through CIP joint.

(b) Maximum Shear and Corresponding Torsion

Maximum shear occurs under full service loads applied to both sides of the beam, corresponding to zero torsional load on the beam. At the critical section [= $h/2$ = 8 in. (203 mm) away from the face of support], the factored shear V_u was found to be 158.6 kips (705 kN). The design shows that an amount of 1.74 in.^{2}/ft (3680 mm^2/m) is needed for the applied shear.[1] Using three #4 U-stirrups with standard hooks around longitudinal reinforcement (see Fig. 17) spaced at 8 in. (203 mm) would be adequate for shear.

Step 4. Transverse Flexural Design

Because of the relatively large beam width and small column width, the beam may be subjected to significant transverse shear and moment. The ultimate shear per unit length V_u = 4.64 klf (67.7 kN/m).

The design shear capacity is:

$$\phi 2\sqrt{f'_c}\,bd = 0.85\,(2)\sqrt{5000}\,(12)\ (2)\ (1.75)/1000 = 5.05 \text{ klf } (73.7 \text{ kN/m})$$

Thus, the top and bottom flanges are capable of transmitting the shear forces to the column line. The transverse moment M_u = 15.1 ft-kips/ft (67.2 kN·m/m). The required steel area is approximately equal to 0.26 in.^{2}/ft (550 mm^2/m). This area is added to the area required for shear and torsion, and is detailed as shown in Fig. 17.

Step 5. Deflection

The deflection calculations of a continuous member were based on a modified version of the procedure proposed by Tadros, et al.[16] The calculated long-term deflection can be shown to be 0.37 in. (9 mm), which is less than $L/480$ = 0.90 in. (23 mm), the maximum allowed by the ACI Code for members attached to nonstructural elements likely to be damaged by large deflections.

Step 6. Beam Ledge Design

Design of the beam ledge is performed according to Section 4.7 of Reference 17. The full gravity loads on each side of the beam introduced by the hollow-core slabs are 2.03 klf (29.6 kN/m), superimposed dead load is 0.59 klf (8.6 kN/m), and live loads are 0.83 klf (12.1 kN/m). The factored shear, V_u, is equal to 3.65 klf (53.3 kN/m) which is to be resisted by #3 closed stirrups spaced at 8 in. (200 mm), one #3 bar at the top corner, and one ½ in. (13 mm) diameter strand from the flexural reinforcement.

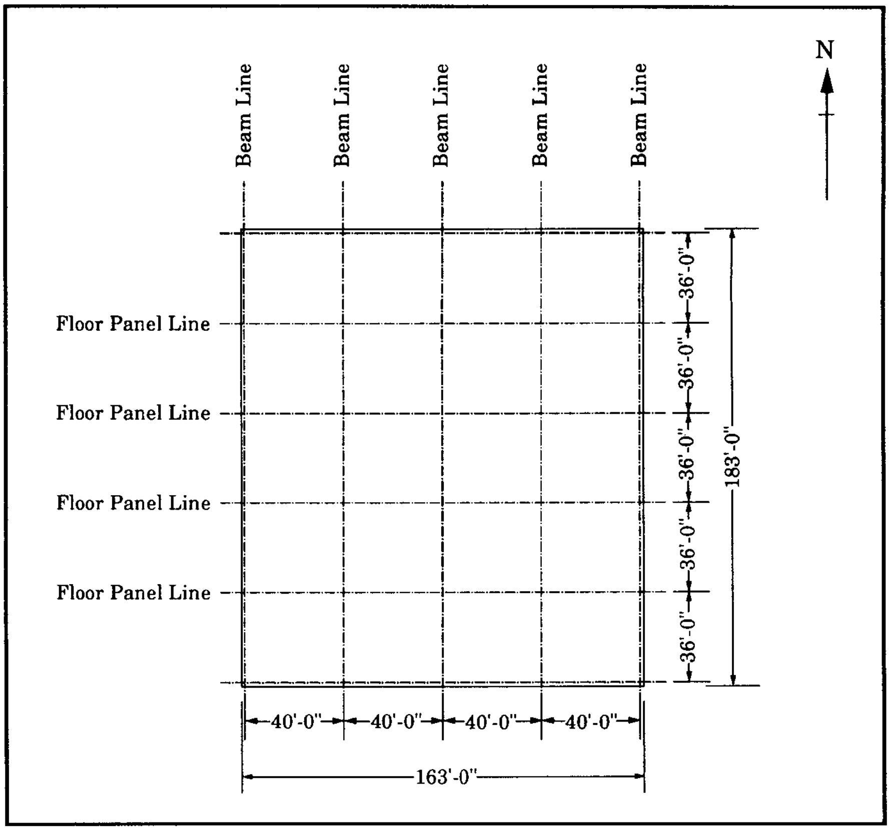

Fig. 19. Floor plan.

Design Summary

Longitudinal Reinforcement

Preliminary and detailed flexural design in Steps 1 and 2 indicate that 24 ½ in. (13 mm) diameter 270 ksi (1860 MPa) low-relaxation strands,

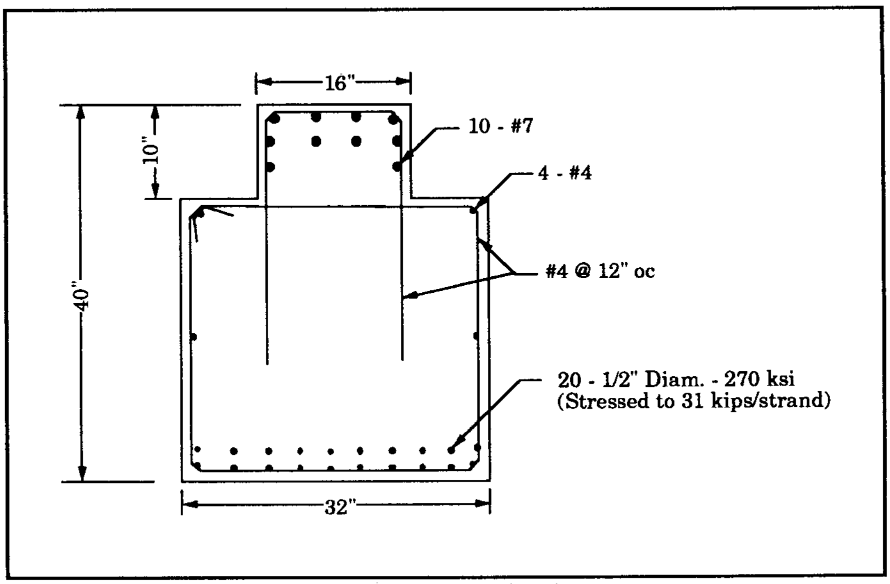

Fig. 20. Reinforcement details — Conventional framing system.

prestressed to 175 ksi (1210 MPa), and 26 nonprestressed ½ in. (13 mm) diameter strands are needed for the positive and negative moment requirement, respectively. The positive moment strands are bent at 90 degrees at the CIP joint to account for possible moment reversal if the system is used for lateral load resistance (see Fig. 18).

To satisfy the working stresses at the top of the end section at prestress release and longitudinal steel requirement for combined shear and torsion, six #6 mild steel bars are provided. These bars are spread around the perimeter of the beam. From Step 6, an additional #3 longitudinal bar at the top outer corner of the ledge is adequate. The selected longitudinal reinforcement is shown in Fig. 16.

Transverse Reinforcement

The transverse reinforcement consists of shear, torsional, transverse flexural and beam ledge reinforcement. Steps 3 and 4 show that a reinforcement area of 0.113 in.^{2}/ft (240 mm^2/m) is needed for torsion, 1.74 in.^{2}/ft (3680 mm^2/m) for shear, and 0.26 in.^{2}/ft (550 mm^2/m) for transverse flexure. For the ledge, #3 bar at 8 in.

Table 2. Cost comparison.

Items	Unit price	Conventional system Quantity	Conventional system Cost ($)	Proposed system Quantity	Proposed system Cost ($)
(1) Interior beam					
(a) Concrete	$54.00/yd^3	307.84 yd^3	16,620.00	241.28 yd^3	13,030.00
(b) Strands	$0.20/ft	20,800 ft	4,160.00	35,256 ft	7,050.00
(c) #4 Reinforcing bars	$0.15/ft	10,910 ft	1,640.00	23,688 ft	3,550.00
(d) #7 Reinforcing bars	$0.43/ft	10,400 ft	4,470.00	5,200 ft	2,240.00
(e) Additional field cost					
CIP concrete	$54.00/yd^3			18.96 yd^3	1,020.00
#6 Reinforcing bars	$0.32/ft			1,152 ft	370.00
#4 Ties	$0.15/ft			2,496 ft	370.00
Field concrete form	$300.00/yd^3			18.96	5,690.00
(f) Connection	$5.00/conn.	600	3,000.00	600	3,000.00
(g) Labor	$9.00/man-hr	2,080 man-hr	18,720.00	3,120 man-hr	28,080.00
Subtotal		48,610.00		64,400.00	
(h) Overhead and profit	Estimate 45 percent		21,870.00		28,980.00
(2) Exterior beams	$50.00/ft	693 ft	34,650.00	693 ft	34,650.00
(3) Columns	$70.00/ft	890 ft	62,300.00	770 ft	53,900.00
(4) Hollow-core slabs					
10 in. hollow-core	$3.75/ft^2	56,092 ft^2	210,350.00		
8 in. hollow-core	$3.00/ft^2			48,356 ft^2	145,070.00
(5) Exterior wall system	$20.00/ft^2	21,359 ft^2	427,180.00	18,603 ft^2	372,060.00
(6) Difference in shear wall, stairs, elevators walls, etc.				$5,000.00	
Total cost		809,960.00		699,060.00	
Building gross area (precast concrete only)		59,658 ft^2		59,658 ft^2	
Cost/ft^2 (precast concrete area only)		$13.57/ft^2		$11.72/ft^2	

Metric (SI) conversion factors: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 sq ft = 0.093 m^2; 1 cu yd = 0.7646 m^3.
Savings = $809,960.00 - $699,060.00 = $110,900.00 or 13.7 percent.

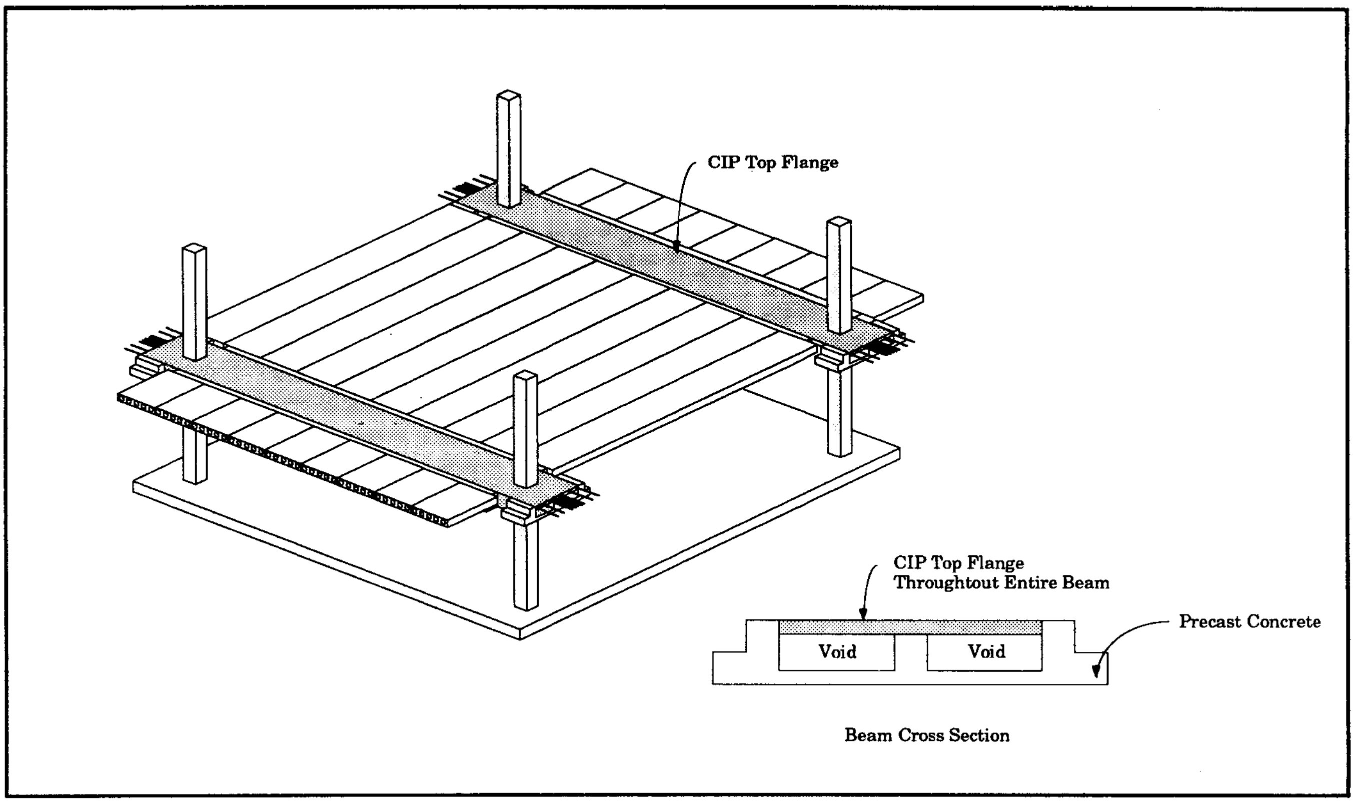

Fig. 21. Possible Variation 1.

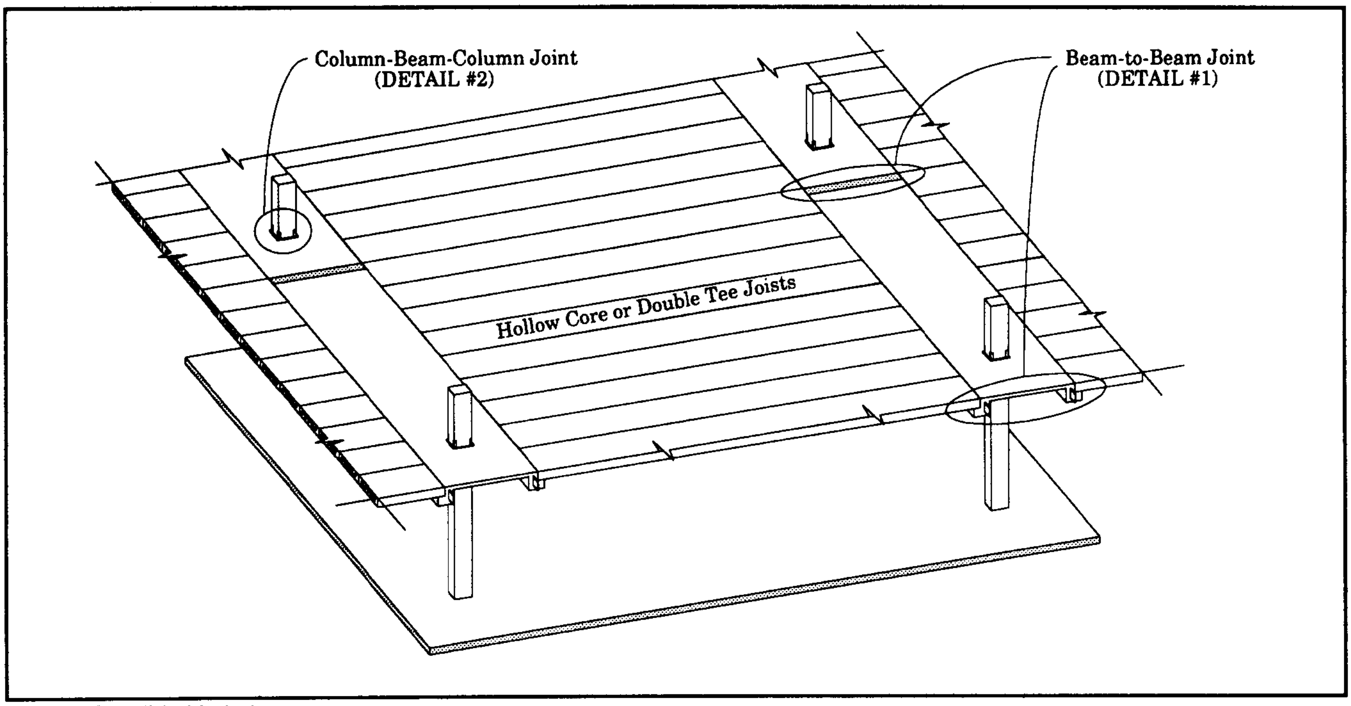

Fig. 22. Possible Variation 2.

(200 mm) is adequate. The selected reinforcement is detailed as shown in Fig. 17.

Step 7. Connection Details

(a) Temporary Connection

The temporary connection consists of steel angles and threaded rods (Fig. 18). These components are subjected to one-half of the total beam weight equal to 17 kips (76 kN) and construction loads equal to 2.88 kips (13 kN) from each side of the column. Assuming that these loads act at a distance of 2.3 in. (58 mm) from the angle face, the corresponding moment to be resisted by the angle is 55.7 in.-kips (6.3 kN·m).

Using a 6 ft (1.8 m) long 6 x 4 in. (152 x 102 mm) angle, the minimum thickness required was found to be 0.415 in. (11 mm). Thus, a 6 x 4 x ½ in. (152 x 102 x 13 mm) 6 ft (1.8 m) angle on each side of the column is adequate. For additional safety, a 5/16 in. (8 mm) thick stiffener is added to the angle. Two 1¼ in. (32 mm) diameter threaded rods were used to attach the angles to the column.

(b) Reinforcement at CIP Joint

The load applied to the outside webs of the beam needs to be transferred safely to the column support. The top and bottom flange of the beam can be shown to have adequate shearing

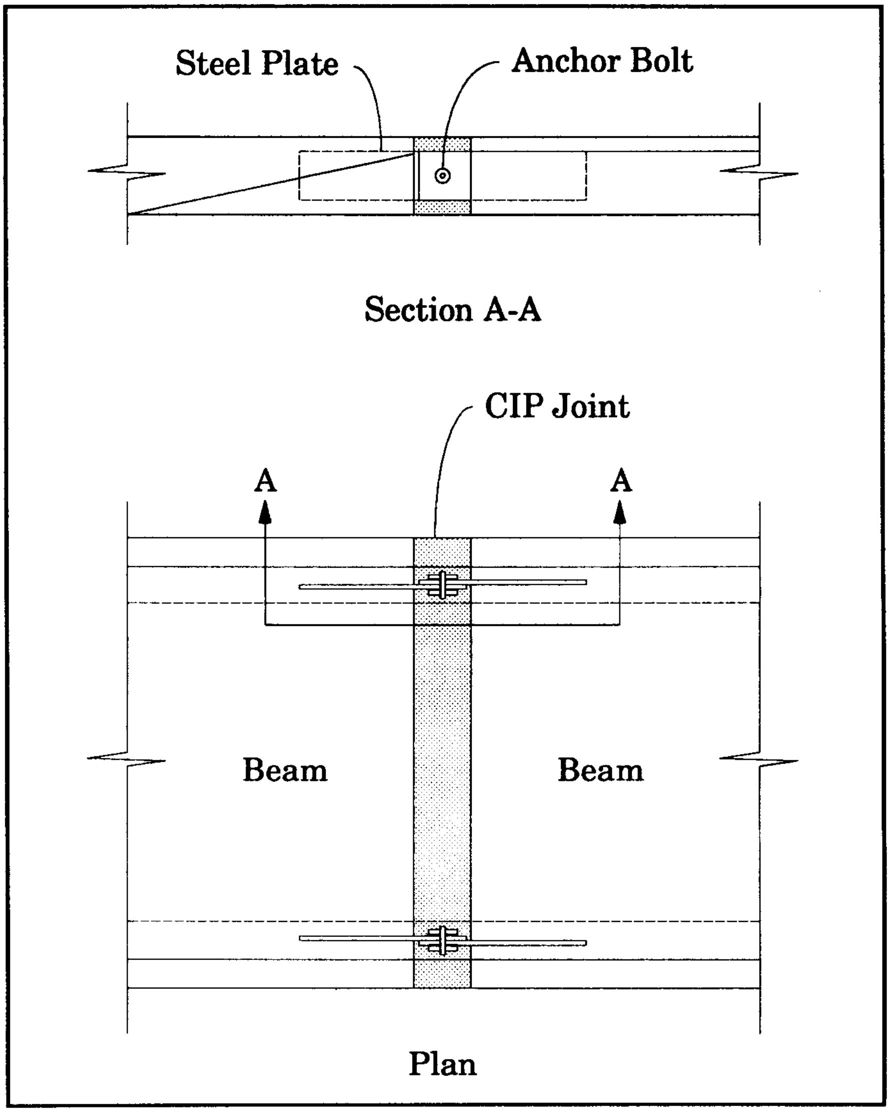

Fig. 23. Section and plan for Detail 1.

capacity in the transverse direction. Also, the transverse reinforcement in the top flange was designed to resist the transverse moment (see Step 4). However, as an added safety measure, the transverse "beam" at the columns should be designed to carry some of that load.

Due to the relative stiffness of the beam and to its proximity to the column support, it was decided to provide an additional amount of steel equal to one-third of the total transverse moment reinforcement. This is equal to:

$\frac{1}{3}(36)(0.26) = 3.12$ in.^{2} (2010 mm^2)

Thus, providing three #10 bars at the top of the "beam" would be sufficient. A more accurate analysis would involve a highly indeterminate two-way model, e.g., a grid model. Design to give a shear capacity at least equal to the bending capacity is conducted. The corresponding stirrups are #3 bars at 6 in. (152 mm).

(c) Two-way (punching) shear

The shear strength of the concrete around the column area is checked against Section 11.12.2.1 of the ACI 318-89 Code.[9] The maximum factored shear from beam weight, hollow-core slabs, superimposed dead load, and live load acting on an area of 36 x 40 ft (11.0 x 12.2 m) was found to be equal to 343 kips (1530 kN).

From the three quantities stated in Section 11.12.2.1 of the ACI 318-89 Code,[9] the quantity $v_c \leq 4\sqrt{f_c'}\,b_o d$ was found to be the controlling quantity. Setting the value ϕV_c against the total factored shear, it could be shown that a concrete strength of 2750 psi (19.0 MPa) is adequate for the applied loads. Thus, the section is adequate for two-way shear. Its strength is further enhanced by provision of the transverse steel cage.

ECONOMIC ANALYSIS

An economic analysis is given below for a two-story office building located in Omaha, Nebraska. The building has a typical bay size of 36 x 40 ft (11.0 x 12.2 m) (see Fig. 19). It is required to evaluate the cost of using the proposed 8 ft x 16 in. (2440 x 410 mm) inverted tee beam and compare the total cost of this system with the cost of a system using an equivalent capacity 32 x 40 in. (0.8 x 1.0 m) conventional inverted tee beam.

Assume that the minimum ceiling height is 9 ft (2.7 m) and that an additional 2 ft (610 mm) duct space is provided for the H.V.A.C. system. Thus, the floor-to-floor height for conventional framing is:

9 ft + 2 ft + 40 in. = 14 ft 4 in. (4.4 m)

and for the proposed system it is 12 ft 4 in. (3.8 m). Results of the structural designs for both systems are shown in Figs. 16, 17 and 20.

The bottoms of columns are assumed to be 1 ft (305 mm) below grade. Detailed analysis of items that are similar to both systems are ignored. For example, a rough estimate cost is used for exterior beams where wide beams are not necessary because head room is not critical. For cost comparison, hauling and erection costs are ignored since both systems involve the same number of pieces.

In order to reflect a true comparison of F.O.B. plant costs, only the cost of new items not normally needed in a

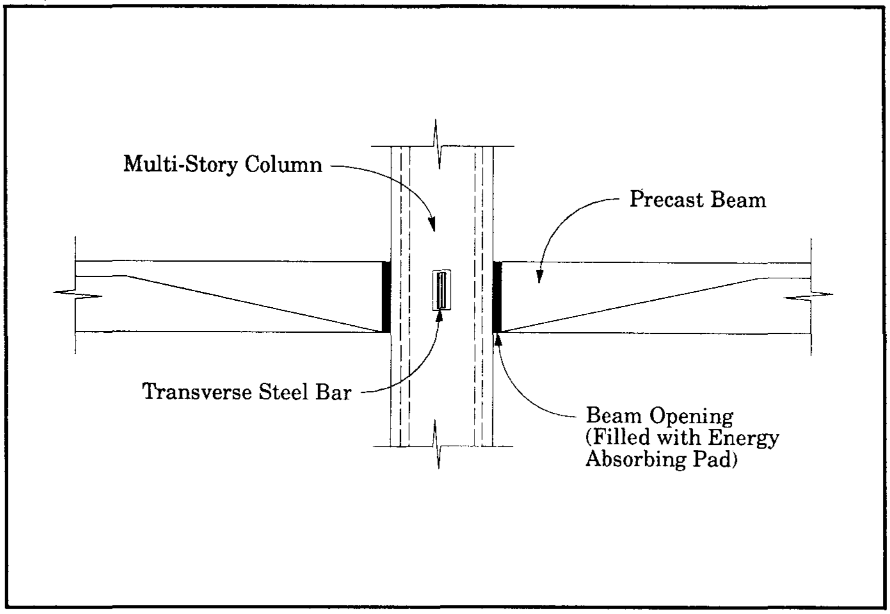

Fig. 24a. Detail 2 — Multistory column construction.

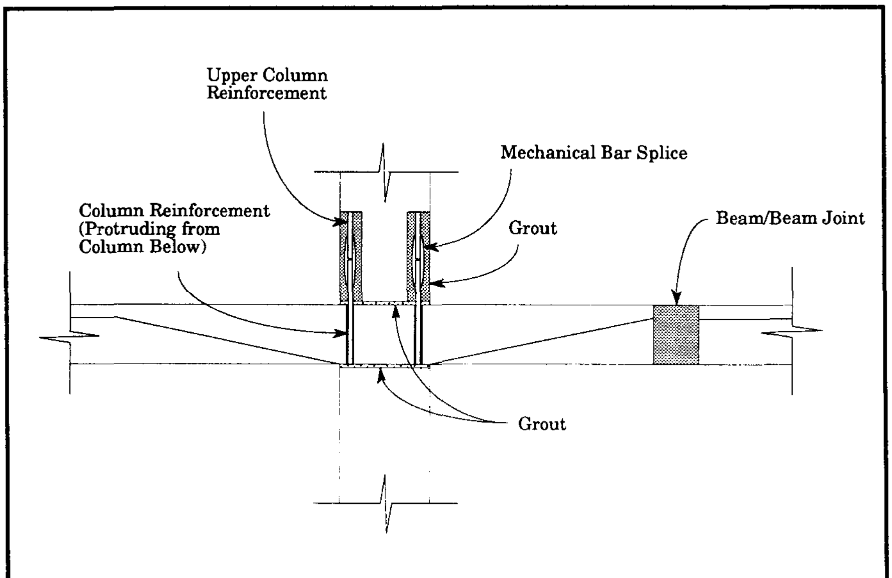

Fig. 24b. Detail 2 — Single-story column construction.

conventional system during erection are included — for instance, CIP concrete between beams and field placed reinforcement.

Initial form costs are neglected because these costs depreciate over the years. Thus, any differences in the initial form costs will be insignificant. Also, the cost of angles and threaded rods for temporary beam support is ignored since it can be amortized over a large number of uses.

The itemized cost comparison of the two systems is shown in Table 2. As indicated in the table, although the cost of the 8 ft x 16 in. (2440 x 410 mm) beam is higher than a conventionally designed 32 x 40 in. (813 x 1020 mm) inverted tee beam, the overall cost of the proposed system has a more than 13 percent savings.

It is estimated that the savings will be higher for structures over two stories high. Finally, savings could also be realized from the reduction in shear wall needs for lateral load resistance and in foundation needs due to the reduced lateral forces as a result of reduced building height.

HOUSING OF UTILITIES

One potential use of beam voids is for the housing of electrical and mechanical systems. This concept was examined for the authors by Maniktala Assoc. of Lincoln, Nebraska. The concept is found workable, and it is explained in detail in Reference 1.

POSSIBLE VARIATIONS

Two possible variations of the proposed system have recently been investigated. A preliminary investigation shows that they hold good promise. The first one is a system which uses a precast concrete beam with a CIP concrete top flange throughout the entire beam (Fig. 21). Thus, only the three webs and the bottom flange of the beam are precast. The top flange is formed with CIP concrete in the field. Similar to the proposed system, 2 ft (0.61 m) on each side of the column in the beam line direction is filled solid with CIP concrete.

Another possible variation of the system is a "dry" system as shown in Fig. 22. The beams are spliced at a distance 5 ft (1.5 m) away from the face of the support with bolted connections (see Fig. 23). Steel plates are embedded in both ends of the beam, and these plates are connected together with anchor bolts. A 1 ft (0.305 m) strip of CIP concrete joint is used to cover the steel plates and bolts for fire and corrosion protection.

The three main features of this system are: (a) CIP is not required during erection, (b) beam forming is simplified and (c) beams are pretensioned for both positive and negative moments.

Two types of construction are developed for this system, namely multistory and single-story column construction.

In multistory column construction, an oversized opening is made in the beam to allow the column to run through continuously (see Fig. 24a). The gap between the beam and the column is filled with energy absorbing material. A steel bar is inserted through the column in the beam transverse direction to transfer loads into the column.

In single-story column construction, however, the beam runs uninterrupted through the column (see Fig. 24b). Sleeves are made in the beam to allow room for column reinforcement to protrude through. The column-to-column connection is achieved using mechanical bar splices. Pockets in the column and sleeves in the beam are grouted after the splicing. A layer of grout is provided both under and over the beam at the bearing area.

An elevation and cross sections of the beam are shown in Fig. 25. As shown in the illustration, the thickness of the top flange increases from 3½ in. (90 mm) at 5 ft (1.5 m) away from the face to a full depth of 16 in. (410 mm) at the face of the support.

CONCLUDING REMARKS

Some of the commonly used precast concrete office building systems are presented in this article. These include American, Australian and European systems.

A new system developed by the authors is described. It utilizes an 8 ft (2.4 m) wide, 16 in. (410 mm) thick

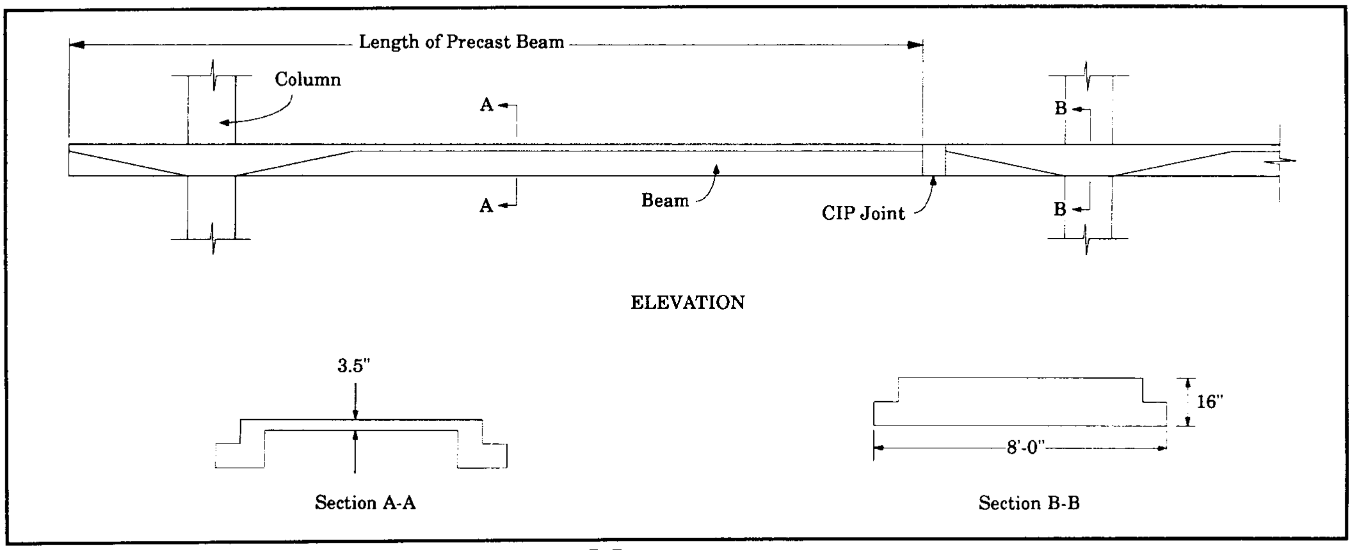

Fig. 25. Beam elevation and cross sections at A-A and B-B.

inverted tee beam supporting hollow-core slabs. Voids are created in the beam to improve its structural performance and to reduce the total weight of the system.

The beam is made continuous under superimposed loads. Continuity in the beam negative moment area is achieved using field placed top reinforcement and top flange concrete in the column vicinity.

Column corbels are fully eliminated to simplify production and reduce cost. In their place, the column concrete at the beam level is blocked out in the plant and replaced with CIP concrete. The proposed system does not require any construction shoring or field welding. Temporary beam support at the columns consists of steel angles and threaded rods.

Various production and erection details are given. Of special interest are several possible techniques for void forming in the inverted tee beam.

A design example of the beam is worked out. Results of the design show that an 8 ft (2.4 m) wide, 16 in. (410 mm) thick beam is adequate for a 36 x 40 ft (11.0 x 12.2 m) interior bay with typical office floor loading.

The economy of the new system is illustrated with an example of a two-story, 163 x 183 ft (49.7 x 55.8 m) building. It is shown that a more than 13 percent savings can be realized by using the proposed system instead of a conventional 32 in. (810 mm) wide, 40 in. (1020 mm) thick inverted tee beam. The concept of incorporating mechanical and electrical systems into the structural system is addressed.

Near its conclusion, this research has produced two variations of the proposed inverted tee beam. The first one involves a full CIP top flange, rather than only the two outside portions. The second variation allows an "all dry" structural frame; the beam would not require void forming, nor would it require CIP concrete during erection. This would save valuable crane time and assure the high quality of precast concrete throughout the structural frame.

Finally, it is suggested that a full-scale test of the proposed system be conducted with special emphasis on the beam-to-column joint.

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REFERENCES

1. Low, S. G., "Minimization of Floor Thickness in Multistory Buildings," Thesis presented to the University of Nebraska-Lincoln, Omaha, NE, 1991, in partial fulfillment of the requirements for the degree of Master of Science.
2. *PCI Design Handbook – Precast and Prestressed Concrete*, Third Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1985.
3. *Composite Dy-Core Office Structures*, Brochure, Finfrock Industries, Inc., Orlando, FL.
4. *The Filigree Wideslab Method of Concrete Deck Construction*, Brochure, Mid-State Filigree Systems, Inc., Cranbury, NJ.
5. *The Composite Prestressed Concrete Framing System* — PSI Brochure, Prestressed Systems, Inc., Miami, FL.
6. *Quickfloor Integrated Building Systems*, Brochure, Quickfloor Co., Portola, CA, 1990.
7. Fogarasi, G., *Prestressed Concrete Technology*, Akademiai Kiado, Budapest, Hungary, 1986, pp. 256-264.
8. *BVM-TIP Building Systems* (in Hungarian), Catalogs, V. 1 and 2, BVM Concrete and Reinforced Concrete Works, Budapest, Hungary, 1989.
9. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-89)," American Concrete Institute, Detroit, MI, 1989.
10. *Seismic Design and Detailing for Spancrete Precast Diaphragm System*, Spancrete of California, Irwindale, CA, pp. 44.
11. *Sweet's Catalog File — Products for General Building and Renovation*, V. 2, Mc-Graw Hill Co., New York, NY, 1986, Section 3.5.
12. Nawy, E. G., *Prestressed Concrete – A Fundamental Approach*, Prentice Hall, Englewood, NJ, 1989, pp. 159-188.
13. Skogman, B. C., Tadros, M. K., and Grasmick, R., "Flexural Strength of Prestressed Concrete Members," PCI JOURNAL, V. 33, No. 5, September-October 1988, pp. 96-123.
14. *PCI Design Handbook – Precast and Prestressed Concrete*, Second Edition, Precast/Prestressed Concrete Institute, Chicago, IL.
15. Zia, P., and McGee, W. D., "Torsion Design of Prestressed Concrete," PCI JOURNAL, V. 19, No. 2, March-April 1974, pp. 46-65.
16. Tadros, M. K., Ghali, A., and Meyer, A. W., "Prestress Loss and Deflection of Precast Concrete Members," PCI JOURNAL, V. 30, No. 1, January-February 1985, pp. 114-141.
17. PCI Committee on Connection Details, *Design and Typical Details of Connections for Precast and Prestressed Concrete*, Precast/Prestressed Concrete Institute, Chicago, IL, pp. 4.24-4.26.