Emulating Cast-in-Place Detailing in Precast Concrete Structures

Reported by Joint ACI-ASCE Committee 550

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This report provides engineers with a practical guide for detailing precast concrete structures that should meet building code requirements in all seismic regions by emulating cast-in-place reinforced concrete design. This report also provides information that shows how emulative precast concrete structures can address any or all of the provisions in accordance with ACI 318-99, including those of Chapter 21, if special attention is directed to detailing the joints and splices between precast components.

Keywords: ductility; elastic design; emulation; flexural strength; joint; precast concrete; precast detailing; reinforcement.

CONTENTS

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CHAPTER 1—INTRODUCTION

Emulative detailing is defined as designing connection systems in a precast concrete structure so that its structural performance is equivalent to that of a conventionally designed, cast-in-place, monolithic concrete structure (Ericson and Warnes 1990).

Emulative detailing is different than jointed design where precast elements are separated from each other but are connected with special jointing details like welded or bolted plates. As commonly applied, the term "emulation" refers to the design of the vertical or horizontal elements of the lateral-force-resisting system of a building. Emulative detailing of precast concrete structures is applicable to any structural system where monolithic reinforced concrete would also be appropriate, regardless of seismic region (Precast/Prestressed Concrete Institute 1999).

Design practice in some countries with a high seismic risk, such as New Zealand and Japan, follow design codes that address precast concrete designed by emulation of cast-in-place concrete design. Performance of joints and related details of emulative precast concrete structural concepts have been extensively tested in Japan. Because emulative precast concrete structures have been constructed there for over three decades, emulative methods for seismic design are widely accepted. Until recently, this practice has not been formally followed in the U.S.

Typical details showing proportional dimensions, as well as reinforcing steel, are schematic only and are provided solely to demonstrate the interactivity of the jointing essentials. All connection details will be subject to structural analysis and compliance with contemporary code requirements. At the time of this writing, splicing reinforcing bars by welding or lapping was not permitted by code whenever the bars were subjected to stresses beyond the actual yield points of the reinforcing steel being used. According to certain tests of mechanical splices reported by the California Department of Transportation (Noureddine, Richards, and Grottkau 1996), concern was expressed about staggering of mechanical splices of reinforcing bars. Staggering is not required by current and developing codes.

Only reinforcing bar details essential to make the illustration more understandable are shown to avoid congestion and provide clarity. Other reinforcing steel that would typically be incorporated into a conventional design is intentionally not shown. The specification and delineation of reinforcing bars or strand sizes and locations, layers, types, and numbers is the responsibility of the designer.

CHAPTER 2—GENERAL DESIGN PROCEDURES

A large body of technical information is available for the design of cast-in-place reinforced concrete structures, and extensive research and development is on-going for all types of cast-in-place concrete technology. Numerous textbooks have been written about the behavior and design of cast-in-place reinforced concrete. Design procedures and examples for cast-in-place reinforced concrete are available (Cole/Yee/Schubert and Associates 1993). Building codes are regularly revised to reflect new research and technology developments, and the results are incorporated into teaching and working practice (Uniform Building Code; ACI 318). This knowledge for designing reinforced cast-in-place concrete structures is readily applicable to the design of emulative precast concrete.

The analysis and design of cast-in-place reinforced concrete structures is based on the premise that the entire system behaves monolithically as a unit. A cast-in-place concrete structure is actually built section by section with joints between the concrete placements because of limitations in concrete placing, construction procedures, or both. Due to the continuity of the reinforcement and specific requirements for construction joints, the structure performs as a unit. The principal element of the emulative detailing of precast concrete is to detail a precast structure that will exhibit structural behavior similar to that of a cast-in-place structure.

Construction joints, whether in prefabricated or cast-in-place concrete structures, should be located and detailed to ensure transmission of induced forces and loads in both the concrete and reinforcing steel. For precast concrete, emulative construction joints will likely occur at the same locations as dry joints in the structural elements. Joints will usually be located at the ends of beams and columns, at both the ends and sides of floor elements, and at the joints between wall elements.

The essential differences between cast-in-place reinforced concrete and emulative, reinforced, precast concrete relate to field connections and assembly of the prefabricated elements. Prefabricated elements have additional design requirement for stripping, transportation, and erection loads imposed on them, but the structural analysis and element design is essentially the same for both types of construction.

Using emulative methods for connecting precast concrete elements, the detailing process will follow three general steps:

1. The desired structural system for resisting gravity and lateral loads is selected. A separate gravity-load-resisting frame can be combined with lateral-load-resisting shear walls, or both functions can be accomplished with moment-resisting frames. System selection is often controlled by the height of the building and the span of the components as well as architectural requirements.

2. Design and detail the structure to meet the requirements of the applicable building code as if it is to be constructed of monolithic cast-in-place reinforced concrete, keeping in mind that the structure will be divided into structural elements of sizes and shapes that:

- Are suitable for plant fabrication;
- Are capable of being transported; and
- Can be erected by cranes available to the contractor.

3. Organize the structure on paper into typical precast elements of appropriate sizes and shapes to meet the foregoing criteria. Then design and detail the appropriate connections to satisfy the requirements of the applicable building code to

allow the precast elements to be reconnected in a way that emulates a monolithic system.

The manufacture and construction of precast structures will normally follow five steps:

1. Manufacture the precast structural elements with code-compliant mechanisms for splicing the structural reinforcing bars to provide continuity of the reinforcement throughout the structure;

2. Transport the prefabricated elements to the project site if they are cast offsite;

3. Erect and temporarily secure each individual precast element;

4. Connect the reinforcing bars between the precast concrete elements by completing the splices;

5. Connect the precast concrete elements with grout or concrete closures; and

6. Reshore horizontal elements as required.

2.1—Selecting a structural system

Selecting an appropriate structural system, such as shear walls, box structures, moment-resisting frames, and dual systems for both lateral and gravity loads, can be the most important step in achieving an economical, structurally sound design. Essentially, four types of structural elements addressed in model codes are used in combination to form complete building systems. Horizontal elements include beams and slabs. Vertical structural elements include walls and columns or combinations of both horizontal and vertical elements, such as cruciform elements. These elements can be combined in various configurations to form commonly recognized lateral-load-resisting systems, such as shear walls and moment-resisting frames. Emulative detailing principles apply to all of them.

With precast concrete, the designer has the option to select only those frames or walls necessary to resist loads under the code requirements. For seismic conditions, the elements of the gravity load frame need only meet the requirements of ACI 318-99, Section 21.9 (frame members not proportioned to resist forces induced by earthquake motions) and the requirement that each precast member be connected to adjacent members. This requirement can impose additional engineering considerations even when using emulation detailing.

2.1.1 *Shear walls*—Shear walls resist forces in the structure parallel to the plane of the wall. Because of the relatively large depth of the wall members in-plane, significant lateral stiffness is provided. Structures that have shear walls as the principal lateral-load-resisting elements usually perform better under earthquake loading than moment frame structures. There were failures in various degrees in six structures of Northridge. Three parking garage structures used precast elements. The other three were in cast-in-place concrete. Shear walls were intact in both systems (Iverson and Hawkins 1994).

There were failures in various degrees in six concrete structures at Northridge. Three of the parking structures used precast elements.

Three were cast-in-place concrete. Two parking garages using PCI-recommended jointing details for double tee floor systems suffered floor diaphragm failures. Shear walls were intact on both.

The International Building Code, IBC 2000, based on the National Earthquake Hazards Reduction Program (NEHRP) (Building Seismic Safety Council 1997) recommended provisions, recognizes two classifications of shear walls. “Ordinary shear walls” are walls designed in accordance with ACI 318 Chapters 1 through 18. This includes Chapter 16 on precast concrete with provisions for structural integrity. Ordinary shear walls are permitted in buildings in seismic performance categories: A, B, and C. These requirements do not include the seismic detailing provisions of Chapter 21. Systems braced with ordinary shear walls are assigned a response modification factor, R, of 4.5 for load-bearing wall systems, and 5 for shear walls bracing a vertical frame.

The second classification of shear walls in the IBC 2000 is “Special Shear Walls.” These walls meet the requirements for ductile detailing included in ACI 318-99, Section 21.6, “Special reinforced concrete structural walls and coupling beams.” Systems braced with special shear walls are assigned a response modification factor of 5.5 for load-bearing wall systems, and 6 for shear walls bracing a vertical frame. Special shear walls are used in buildings in seismic performance categories: D, E, and F. Although not required for regions of lower seismic risk, engineers can design special shear walls for these conditions for their increased integrity, strength, and ductility, and for the reduction of base shears afforded by the higher R factors.

For ordinary precast shear walls, emulation does not provide specific benefit. The level of strength and ductility reflected by the R factors only requires the standard details used with precast and tilt-up construction. For special shear walls, however, only those walls that meet the ACI 318 Chapter 21 requirements are recognized. Precast walls, then, need to emulate the performance and detailing of monolithic cast-in-place walls using the rules that were developed for cast-in-place construction. At this time, the only alternative to emulation for special shear walls is the general provision of ACI 318-99 Section 21.2.1.5, which allows alternative systems if the proposed system is demonstrated by experimental evidence and analysis to have strength and toughness equivalent to cast-in-place reinforced concrete. For moment frames, the engineer can refer to ACI ITG/T1.1-99, “Acceptance Criteria for Moment Frames Based on Structural Testing.” However, this is not considered emulation, but rather a special procedure to allow newly-developed jointed frame systems.

Although not prescribed explicitly in the codes, provisions do allow for the consideration of soil-structure interaction. NEHRP (Building Seismic Safety Council 1997) includes requirements that permit the consideration of soil-structure interaction in design. These considerations reflect the increased flexibility and damping due to interaction between the foundation and soil continuum. Such interaction may decrease the design values of base shear, lateral forces, and overturning moments, but they may increase the values of the lateral displacements and the secondary forces associated with P-delta effects. When using stiff wall elements, however, the increased displacements may have minimal effect on overall

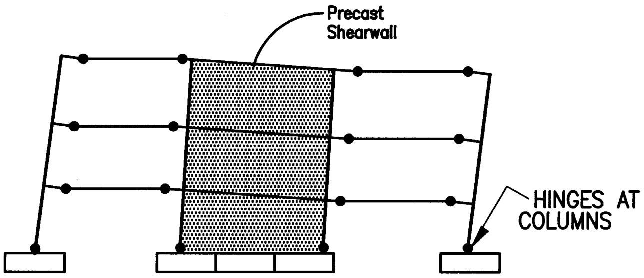

Fig. 1—Dual building with rotation of the shear wall at each floor.

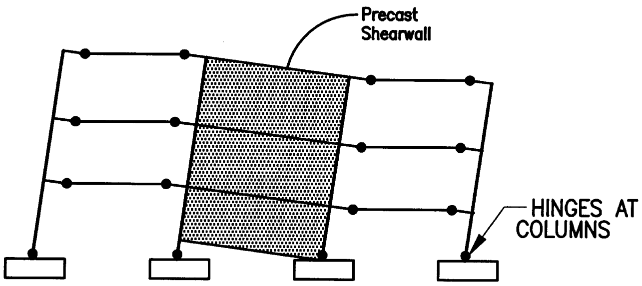

Fig. 2—Dual building, ductile yielding of partially debonded bars between foundation and shear wall boundary elements.

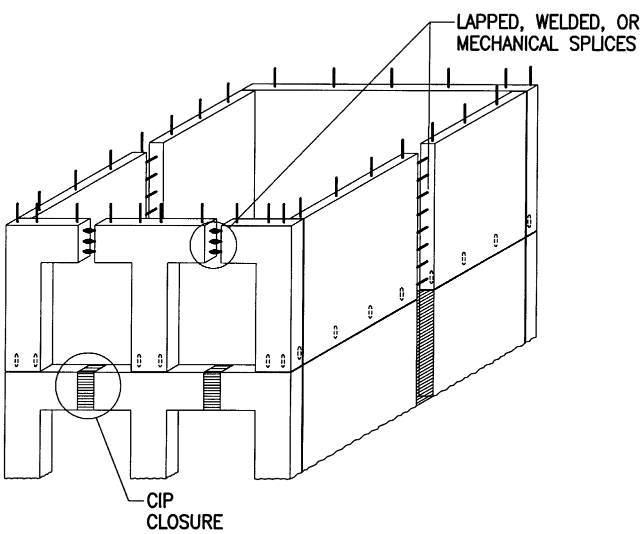

Fig. 3—Precast shear tower using mechanical splices and cast-in-place closure connections between elements.

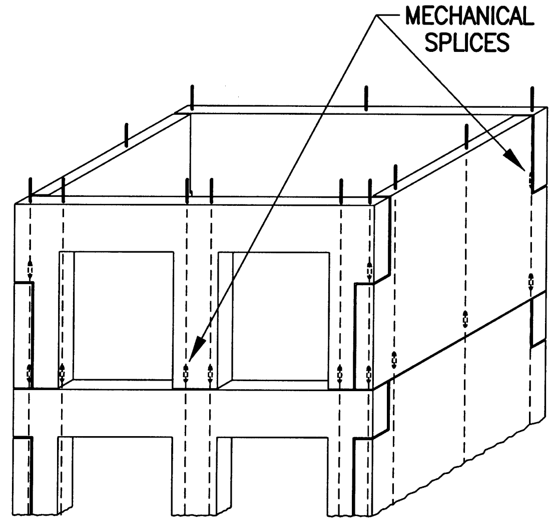

Fig. 4—Use of mechanical connections and interlocking precast wall elements to create a monolithic shear tower. Note: Erection sequencing must be coordinated.

stability. Although the primary mode of inelastic behavior is at the soil/foundation interface, the prescriptive provisions for detailing the structure, which include increased ductility in regions of high seismic risk, are not relaxed.

The desired primary ductile behavior of shear walls emulating cast-in-place detailing is flexural yielding at the wall base and wall joints (Fig. 1). Providing ductility is the intent of the detailing requirements imposed by ACI 318-99, Section 21.6. These include:

- Minimum web reinforcement ratio of 0.0025, unless the design shear force does not exceed $A_{cv}\sqrt{f'_c}$ (where A_{cv} is the gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in.^{2}; and f'_c is the specified compressive strength of concrete, psi). Even if this low shear limit is met, the minimum web steel still has to meet the minimum steel requirements of Chapter 14 for walls;
- Maximum reinforcement spacing of 18 in. (457 mm);
- At least two curtains of reinforcement need to be used in the wall and in the wall-to-foundation interface if the shear force exceeds $2A_{cv}\sqrt{f'_c}$; and
- Continuous reinforcement in walls needs to be anchored or spliced as tension steel.

Because a small rotation in a wall will create a large demand for bar elongation, the ductility at the base is important. Ductility can be increased significantly by debonding bars into and out of the foundation so that they can deform inelastically over a longer length (Soudki, Rizkalla, and LeBlanc 1995), thus resulting in greater nonlinear elongation and rotational ductility (Fig. 2). Reinforcing steel specified for special walls should be ductile and have controlled strength properties. ACI 318-99, Section 21.2.5, requires that reinforcement resisting earthquake forces meet ASTM A 706 with some exceptions.

2.1.2 *Box structures*—Box structures are a special type of building and may fall under the category of walls. Familiar examples of box or cellular structures, shown in Fig. 3 and 4, include stairwells, elevator cores, and panel-type multistory residential buildings. The overlapping corners shown in Fig. 4 provide a strong shear component when completed. In particular cases, when the boxes include integral floors, ceilings, or both, they have been called cells. Even though a large number and variety of buildings falling under this category have been constructed in North America, it has been primarily the Architectural Institute of Japan (AIJ) that has formalized the classification of box structures as a structural system for earthquake-resistant buildings (Suenga 1974).

A box is a three-dimensional cell. Monolithic cells can be emulated by constructing with three-dimensional modules or by assembling with separately manufactured floor and wall panels.

2.1.3 *Moment-resisting frames*—Moment-resisting frames (both steel and reinforced concrete) are used for buildings over a wide range of heights.

There is no technical reason why high-rise, reinforced concrete moment-resisting frames cannot be designed, even to resist large earthquakes, with the intention of having the structure remain elastic. When structures are required to remain elastic, however, elastic design procedures require larger structural members to resist stresses resulting from earthquake loads. This leads to increased material costs as well as higher lateral forces on nonstructural elements, and probable loss of some floor and window opening space due to bulkier columns. Under elastic design provisions, beams may require greater depth, resulting in increased story heights and, consequently, resulting in taller buildings. In regions where relatively minor earthquake loads are expected, elastic design methods can be appropriate when it may not be economical to detail for ductility. The NEHRP-based code provisions permit the use of ordinary moment frames for seismic performance categories A and B.

In June 1978, NEHRP was created. The NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings was first published in 1985 and subsequently updated on a 3-year cycle. These provisions have included not only recommendations for the evaluations of loads and general building details, but also material-specific parameters and detailing provisions that are consistent with those general recommendations.

The 1994 edition of the Recommended Provisions was used in making major changes to the Internal Conference of Building Officials (ICBO) Uniform Building Code (UBC) 1997. The 1997 edition became the basis for the newly merged International Code Council (ICC) International Building Code 2000 model code seismic provisions.

Ductility is an important factor in the design of frame buildings for more severe earthquake regions, such as those constructed in UBC Zones 3 and 4. Buildings in NEHRP seismic performance category C require intermediate moment frames. Buildings of seismic performance categories D, E, and F require special moment-resistant frames (SMRF).

Concrete frames can be readily designed to perform in a ductile manner. Full-scale tests of reinforced concrete beam-column connections have shown that such connections are ductile and can perform effectively under earthquake loading. Plastic hinging of beam-end connections is highly dependent upon the type and amount of reinforcement used in the intended ductile hinge region, usually at or near beam ends.

Chapter 21 of ACI 318-99 provides prescriptive requirements for special moment frames intended to ensure strong column-weak beam behavior.

The AIJ Structural Guidelines for Reinforced Concrete Buildings (Architectural Institute of Japan 1994), a design manual for reinforced concrete frames, explains how to design concrete structures to behave elastically for equivalent earthquake loads associated with horizontal structure accelerations of up to 20% of the gravity. The manual also provides for the deliberate introduction of ductile (inelastic) hinges in the beams near the beam-column junctures and at selected locations in the columns (Fig. 5 and 6). Sufficient ductility and strength are designed into the hinge regions to accommodate lateral accelerations up to 100% gravity. The reinforcement ratio of ductile hinges is intentionally limited so that the bars are capable of being strained significantly beyond their yield point, therefore inelasticly elongating the bars. This mechanism absorbs and dissipates a substantial amount of seismic energy imparted to the frame, and at the same time attenuates the structure's possible tendency to vibrate at the dominant period of the earthquake.

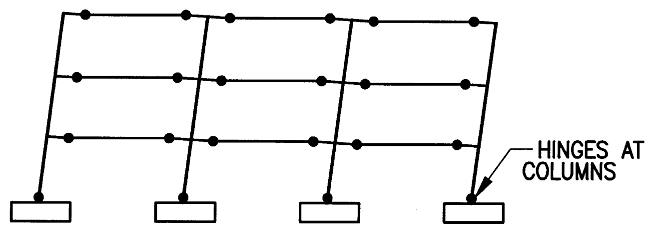

Fig. 5—Planned yield hinges in a ductile moment frame. (Hinges in bottoms of columns of foundations.)

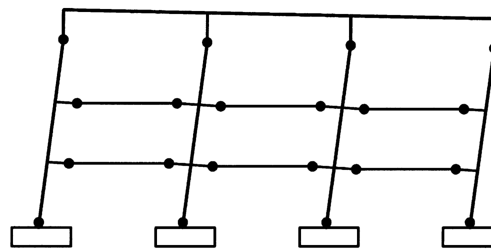

Fig. 6—Planned yield hinges in a ductile moment frame. (Hinges in columns at top and bottom.)

2.1.4 *Dual systems—frames and shear walls*—Dual building systems consist of a combination of shear walls and moment frames. A dual system can be used when a moment-resisting frame alone does not provide sufficient lateral stiffness. Special design attention should be directed to the probable lack of deformation compatibility in both elastic and inelastic modes between frames and walls because they do not deform equally in response to normal as well as severe loads. Connections between frames and walls need to accommodate the different behavior of the two systems.

2.2—Ductility and hinges

Ductility in reinforced concrete frames allows the structure to accommodate large ground motions through energy dissipation at plastic hinge regions. There are also less common events that generate high lateral forces, such as explosions, collisions, and those events associated with high winds, such as tornadoes and hurricanes.

Different levels of ductility can be achieved in reinforced concrete by controlling the primary steel ratio in certain high-moment (high-stress) regions of a member while providing secondary reinforcement for concrete confinement.

These guidelines for structural design of reinforced concrete structures are used in Japan and in other highly active seismic regions of the world.

The AIJ standard requires a structure to have a minimum lateral-load-resisting carrying capacity to limit the response deformation during an earthquake. It also requires the formation of a ductile yield mechanism to dissipate energy from the earthquake; that is, a structural designer should plan a desirable yield mechanism for a structure expected to undergo a design earthquake and then generate such a yield mechanism in the beams during a strong earthquake. Yield mechanisms in moment frames should also be provided between foundations and the base of columns and, under circumstances relating to the amount of acceptable damage to the roof system, at the tops of columns.

Under the AIJ approach, the designer first plans a desirable yield mechanism to give both the required strength to the structure and sufficient ductility to the planned yield hinges (yield-mechanism-design). Next, the designer provides nonyielding regions and members with sufficient elastic strength to encourage the formation of the planned yield mechanism in the intended location of the structure (yield-mechanism-assuring-design). Another feature is a new approach in shear design of members based on a plasticity theorem, in which shear is designed to be resisted by concrete arch and truss mechanisms. This shear design method can be used for beams, columns, and structural walls.

The earthquake resistance of this design approach relies on the energy-dissipation capacity at the planned yield hinges, usually located in beams adjacent to the column faces and in columns and walls at the foundation. Therefore, applying this method is limited to those parts of structures that can develop clearly-defined yield mechanisms.

Because ductility in ordinary (not prestressed) reinforced concrete is mostly a function of the mild steel bars used for reinforcing, a yield mechanism is established in the reinforcement at an intended hinge location to be high enough to exceed the yield point of the steel. This is accomplished by deliberately limiting the cross-sectional area A_s of the steel reinforcement in the intended hinge region, forcing inelastic deformation.

When the natural vibration period of a building, or a harmonic of it, is close to the frequency of seismic waves, the vibration amplitude of the building is reinforced, something like continuously striking a tuning fork. This causes the building to sway back and forth at an ever-increasing extension during the length of time the earthquake continues, the effect being to magnify the intensity of forces. When yield hinges are incorporated into the structure, the yielding of the reinforcement in the hinges dissipates a large amount of energy. This attenuates the natural vibration period of the building so that it cannot resonate in sympathy with the frequency of the earthquake.

2.3—Design and analysis procedures

In general, a building will be classified as a shear-wall structure, moment-frame, or dual system. Preliminary design loads, including seismic-equivalent static-lateral loads, are calculated according to codes and assume the structure to be monolithic cast-in-place concrete. Once the structural elements are preliminarily proportioned, more accurate calculations using Rayleigh's method or a finite-element analysis will frequently result in smaller design loads than those obtained from an initial application of the "equivalent static load" method. The more-accurate loads are then applied to the structural model and the internal design forces are calculated.

2.3.1 *Moment frames*—Analysis of an emulative precast concrete structure follows the same structural analysis procedure as that used for analysis of a cast-in-place reinforced concrete structure.

The required strength of the various components of a lateral-force-resisting system will be determined by the analysis of a linearelastic model of the system. For frames, elastic analysis is used to determine the flexural strength required at the ends of the beams as they frame into the column. To ensure ductile behavior, the steel reinforcement ratio within a ductile hinge region is limited by code to a maximum of 0.025. The positive moment capacity of strength in the beam at the column face has to be at least 50% of the negative moment capacity to resist reversals due to cyclic loading. The balance of the design of the special moment frame, then, is based on making this area the weak link in the frame system.

Columns above and below a joint should have a total flexural capacity M_c that is 20% greater than the sum of the flexural capacity M_g of the beams framing into the joint as provided by ACI 318-99, Eq. (21-1).

$$\Sigma M_c \geq (6/5)\Sigma M_g$$

The requirements for transverse reinforcement in both beams and columns are intended to ensure that the shear strength does not limit the frame capacity and that the areas of yielding are well confined for stable behavior beyond flexural yielding.

2.3.2 *Shear walls*—For walls, simplified analysis methods that rely on the relative shear and flexural stiffness of the walls are available (Precast/Prestressted Concrete Institute 1997). Analysis should consider the effects of shear deformations for walls with aspect ratios lower than 3-to-1. The effects of the eccentricity of the center of mass differing from the center of stiffness of the wall system should be considered along with the code requirement to include 5% eccentricity for accidental torsion. For most precast systems, the stiffness contribution made by connecting the floor to the walls is usually large enough to create moment reversals or fixity in the wall at the floors. Precast walls, then—even those that emulate monolithic construction—should be designed as cantilevered from the foundation.

CHAPTER 3—SYSTEM COMPONENTS

Precast concrete elements are usually produced in a manufacturing plant and then transported to their assigned positions in the building. When detailing the monolithically designed structural elements into discrete precast components, the designer should consider transportation and erection limitations. These limitations include weight (pavement and bridge capacities), height (bridge, tunnel, and underpass clearance), length (maneuverability and state laws), width (permits, escorts, and state laws), and available crane capacities.

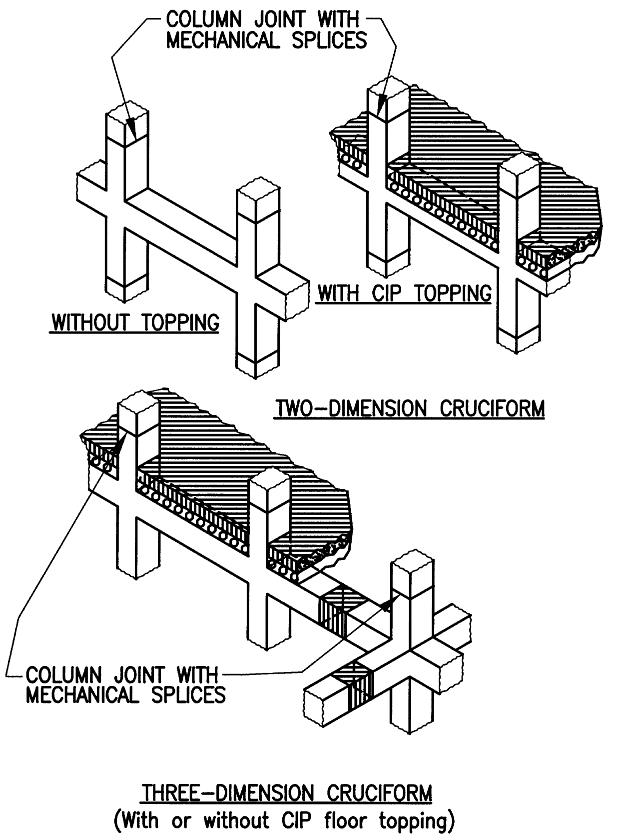

Fig. 7—Typical types of precast concrete "cruciform" elements.

For shear-wall structures, highway bridge-clearance generally restricts panel dimensions. Clearance limitations usually restrict box module heights to approximately one building story. Floor planks and panels are usually narrower than wall panels and a number of pieces can be shipped on each truck.

Beams and columns can be quite long and are usually transported horizontally. H-shaped or cruciform combinations of beam and column members as shown in Fig. 7 can be used to control the location and number of connections in a frame system. The bay size and story height, along with transport size restrictions, will usually control the size of a cruciform subassembly. Cruciform frame elements are sometimes referred to as punched shear walls. They are easy to erect because they can be freestanding and supported with simple braces. All of the connections can be made in regions of low moments.

A key advantage of using cruciform elements is that they permit rapid erection and field assembly of the principal vertical and horizontal structural components of a building, usually with the connections between the precast elements being located in the columns and beams in portions that will experience lower stresses.

Subdividing a structure into components can be achieved most efficiently by working closely with an engineering consultant specializing in precast concrete technology or by consulting with the technical staff of a precast concrete manufacturer. In both cases, the advice of an erector is invaluable. Constraints on available form sizes as well as shipping and handling considerations should be verified with the intended precast concrete manufacturer before proceeding with the design.

CHAPTER 4—CONNECTION OF PRECAST ELEMENTS

Methods to field-connect precast concrete elements should optimize the safety and efficiency of crane and erection crew operations. Because the unit cost of crane time and erection crew time is relatively high, erection scheduling and field connections that use the least amount of time in field assembly can be quite cost-effective. Where ductility is needed, the key element in achieving successful emulation is in selecting field connection details.

Splices for reinforcement used with precast systems that emulate monolithic cast-in-place systems generally involve lapped bars, mechanical splices, and welded splices. When lapped bars are used, the laps need to extend for significant lengths of cast-in-place concrete to permit the lap lengths and confinement hoops required by ACI 318-99, Chapter 21. The cast-in-place section will have to be as long as the required splice length for the bars. In ACI 318-99, mechanical splices are divided into two classifications: Type 1 and Type 2. These mechanical splices are those that meet the requirements of ACI 318-99, Section 12.14.3.2. These splices cannot be used within a distance of two times the member depth from the column or beam face or from sections where reinforcement yielding is anticipated. Type 2 mechanical splices have to develop the specified strength of the spliced bar. The specific requirements for these splices are discussed as follows. Type 2 splices are permitted at any location within a structural element. Welded splices are limited in use similar to Type 1 splices.

4.1—Connections in wall systems

The critical connection in wall systems is usually the connection between the precast panel and the cast-in-place foundation system, because this is the location of maximum shear and moment caused by lateral loads. In tall buildings, other wall panel-to-panel connections can be as important.

Horizontal joints in panel-to-panel connections are usually a combination of grout and spliced vertical reinforcing bars. The grout provides continuity of compressive forces across the joints, and the bars provide continuity for the tensile forces. Figures 8 and 9 illustrate joints where vertical reinforcement is made continuous with lapped bars in conduit or by splicing bars with a threaded coupler. Rapid field erection is permitted by the use of high-strength joints, such as those shown in Fig. 10, where the vertical reinforcement is spliced and grouted with specially designed and code-approved sleeve connectors. At the wall base, and at other joints where bar yielding can occur, these splices should be Type 2 mechanical splices.

A cast-in-place connection can be used between adjacent walls when tall vertical wall panels are used. Alternatives for completing the vertical connection are illustrated in Fig. 11. They feature a cast-in-place closure strip with horizontal interconnecting-reinforcing steel spliced mechanically. The steel can also be lapped if the splice lap length can fit within the closure placement width and the lap splice is in a region of the member permitted by code. Figure 11(a) is used when there is no architectural concern for appearance, such as in elevator shafts, where the walls will be hidden. Figure 11(b) is used where an architectural concrete face is exposed, such as in airport control towers. Figure 11(c) can be used in punched shear walls, such as those used for joining ends of cruciform beams and headers when there is an architectural concrete consideration.

Connections between floor diaphragms and walls are critical if the floor inertial forces are to be successfully transferred to the wall systems. Regardless of the design approach used in sizing and detailing the walls, some engineers feel that the floor diaphragm and its connections should be designed to remain elastic under seismic loading. Therefore, it is desirable to provide a wall-to-floor connection capacity that is appropriate for the capacity of the wall system. Sample details for these connections are shown in Fig. 12 to 14.

The technique of crossing the positive moment steel shown in Fig. 13 provides for structural reinforcement continuity of the diaphragm across the wall and provides much of the shear reinforcement. During construction, the floor slabs are shored where they meet the walls. Therefore, if the slabs are inadvertently not fabricated sufficiently long enough to bear on the walls, the placing of the cast-in-place concrete in the closure strip accommodates the deficiency.

Figure 15 shows vertical wall joints used in high seismic zones in Japan.

4.2—Connections in frame systems

Ideal locations for connections in frame systems are at points where the frame forces, particularly moments, are likely to be at minimum levels. It is natural to select the inflection points as points to break a monolithic system apart and to reconnect as an emulative precast system. The H-shaped and cruciform frame systems shown in Fig. 7 have connections near where the inflection points under lateral loading are likely to occur. Figure 16 shows several horizontal connections. The 1997 NEHRP provisions require that connections, even at

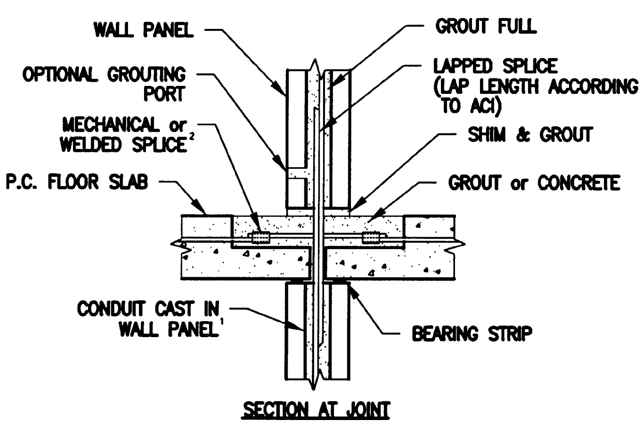

*Fig. 8—Lapped splices in large conduit. ([1]Overlapping bars in grout-filled conduit are extended full-height through the structural element. [2]Welded and lapped splices must be located more than 2*h *(where* h *is floor thickness) from the face of wall. Mechanical splices must be Type 2 if less than 2*h *from face of wall.*

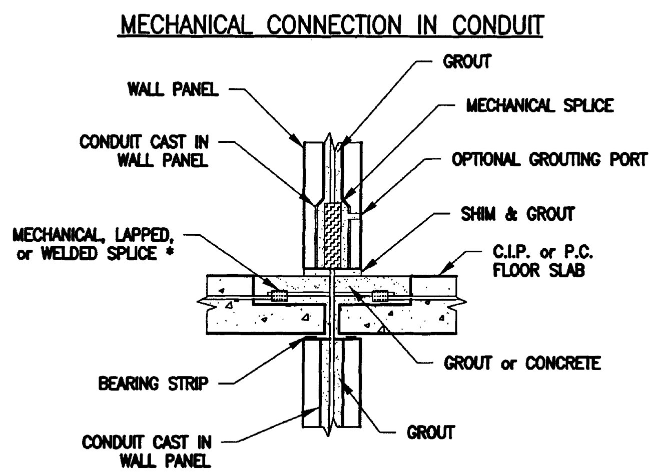

*Fig. 9—Vertical bars in conduit are spliced and the system is grouted. (Procedures: (1) wall panel is erected, but held high; (2) loose vertical bars in the panel being erected are spliced to protruding bars from below; (3) panel is lowered to correct elevation; and (4) conduit is grouted by gravity flow from top or through optional grouting port from bottom of panel.) *Welded and lapped splices must be located more than 2*h *(where* h *is floor thickness) from the face of wall. Mechanical splices must be Type 2 if less than 2*h *from face of the wall.*

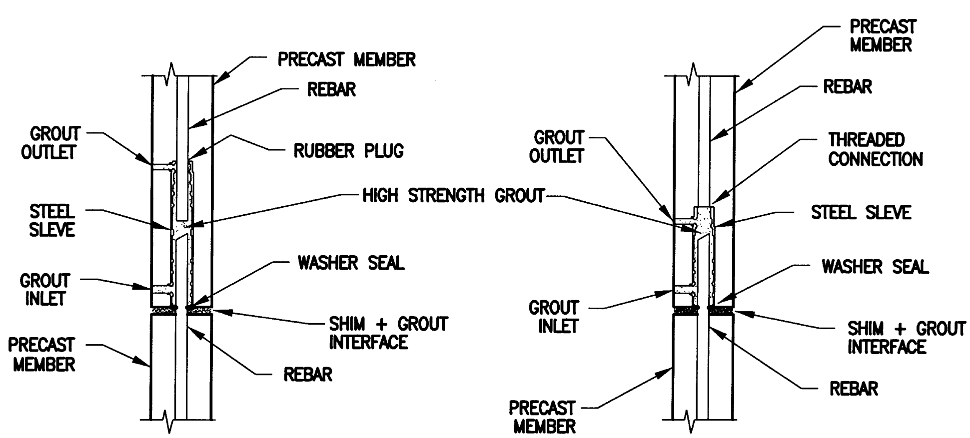

Fig. 10—Typical types of mechanical splices using high-strength non-shrink grout.

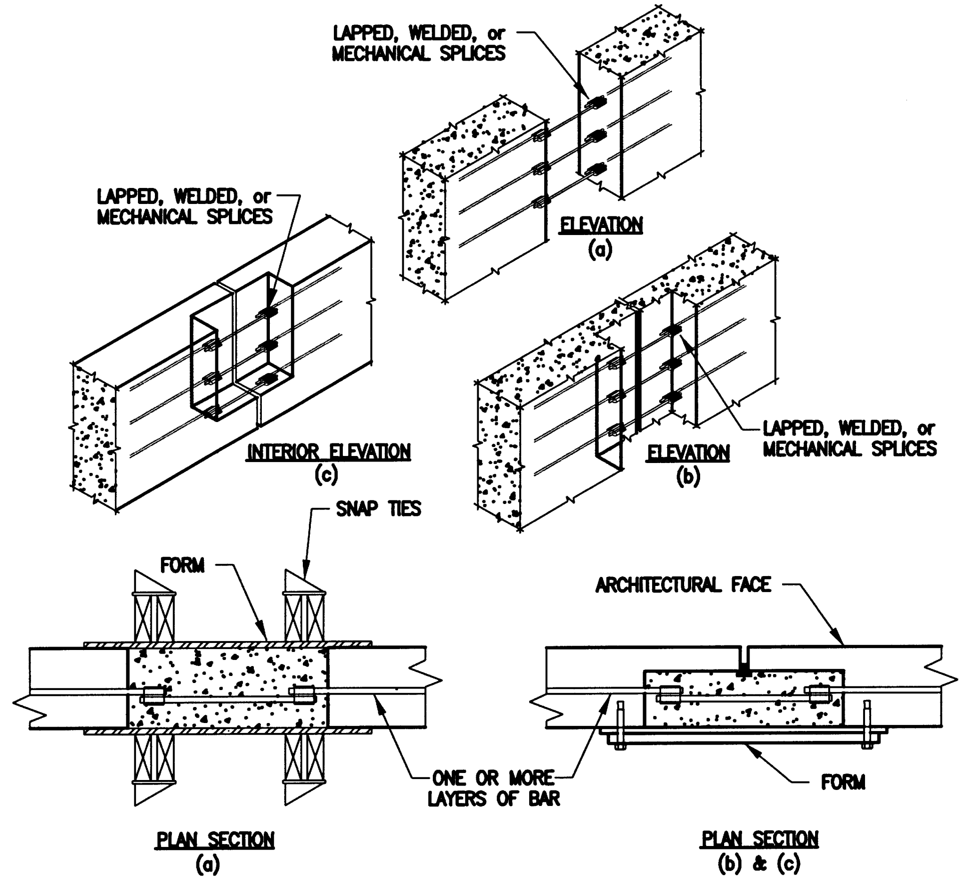

Fig. 11—Variations of splices and cast-in-place closure placements to create vertical joints between precast concrete elements.

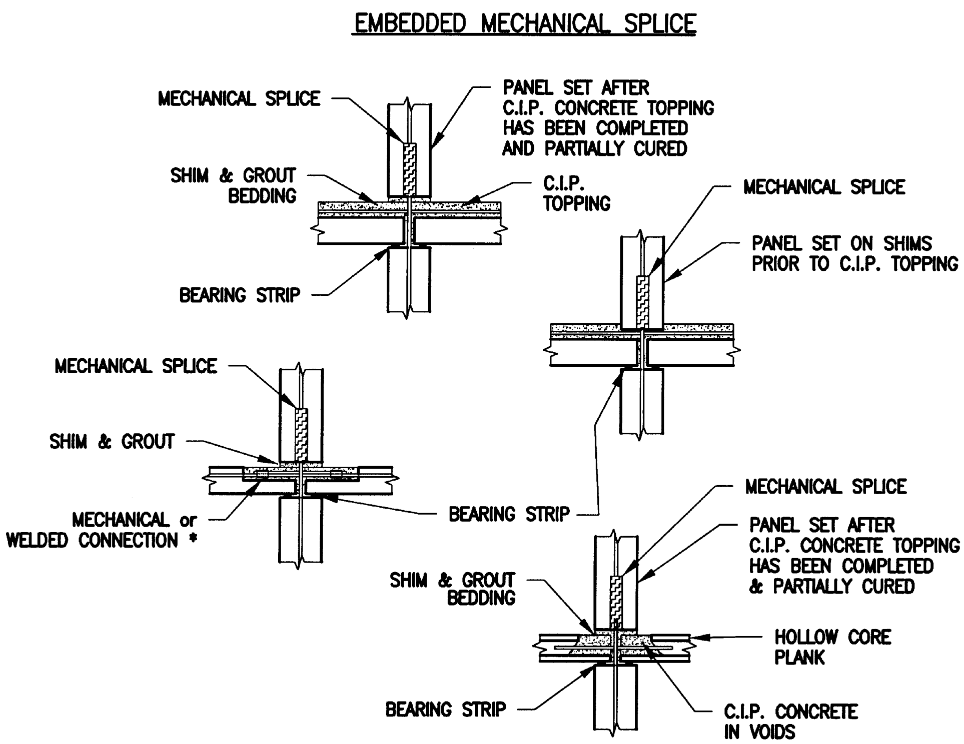

*Fig. 12—Various types of mechanical splice for connection various configurations of precast walls and floors. *Welded and lapped splices must be located more than 2*h *(where* h *is floor thickness) from the face of the wall. Mechanical splices must be Type 2 if less than 2*h *from face of wall.*

nominal inflection points, be designed to provide a moment capacity not less than 40% of the maximum moment.

Figure 17 shows a number of variations of framed connection systems.

For the purposes of fabrication, erection, and transportation, a frame system is often divided into individual beam and column components. Connections of these individual elements can be subjected to large forces and need to satisfy the requirement that the strengths of columns at joints must exceed beam capacities by a specified percentage. Bending moments are usually transferred through these connections by a force couple formed by compression in packed grout or cast-in-place concrete and tension in spliced reinforcing bars. Figures 18 and 19 illustrate types of emulative beam and column joints that can be detailed to accommodate earthquake-generated loads and deformations.

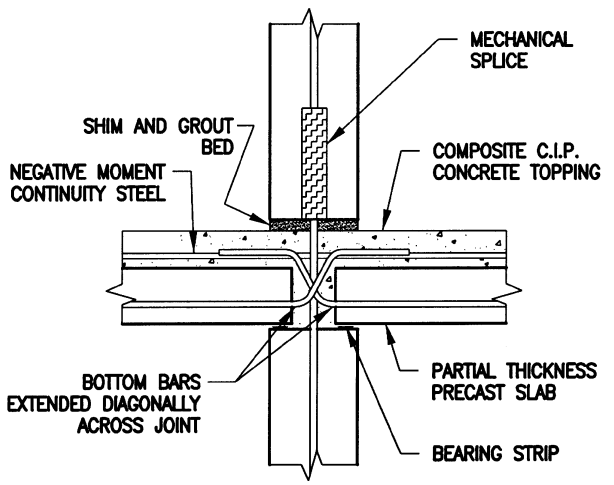

Fig. 13—Floor slab-to-wall detail where diagonal dowels cross the wall joint into the opposite floor.

IBC 2000 permits two methods for frames emulating the behavior of monolithic reinforced concrete. One method uses strong connections (cast-in-place concrete or grout in splices) and complies with all the provisions of Chapter 21, as reviewed previously. The other method permits precast systems that do not meet all the requirements of ACI 318-99, Chapter 21. This method requires the use of strong connections in the most highly stressed portions of the joints that force nonlinear action to occur in the beams away from the joints by a prescribed distance. Section 1908.1.9 of IBC 2000 modifies ACI 318-99 by adding a new Section 21.2.8, which stipulates the following requirements for these systems:

1. The location of the intended nonlinear region is selected to promote development of a strong column-weak beam mechanism under seismic loading. The nonlinear action location can be no closer to the near face of the strong connection than $h/2$;

2. The stresses in the reinforcement in the nonlinear action region are not intended to exceed specified yield outside

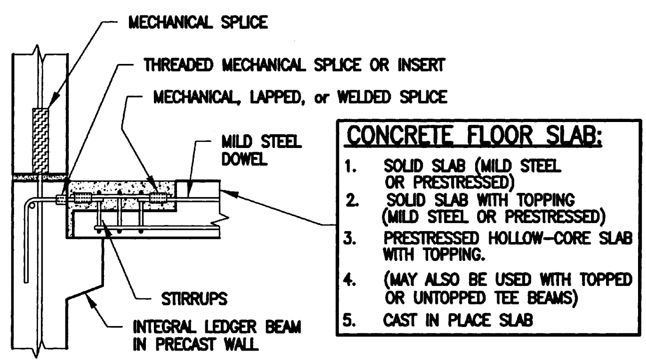

Fig. 14—End detail of a monolithic connection between precast concrete floor element and a precast concrete wall.

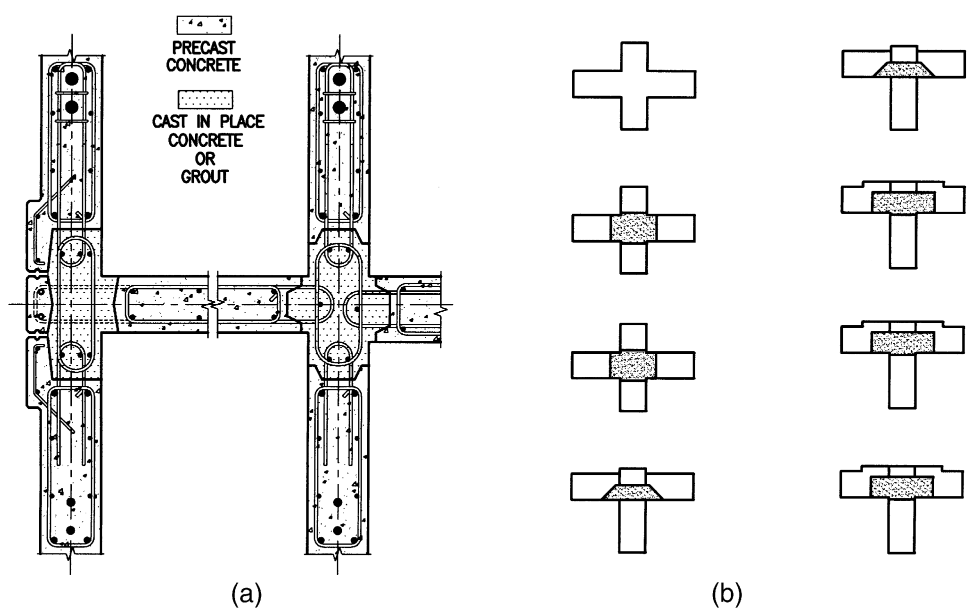

Fig. 15—(a) Plan view of typical grouted or cast-in-place vertical joints in shear wall panels reinforced for high seismic loading (see adjacent plan views for different configurations); and (b) variations of vertical wall-to-wall connections (plan views).

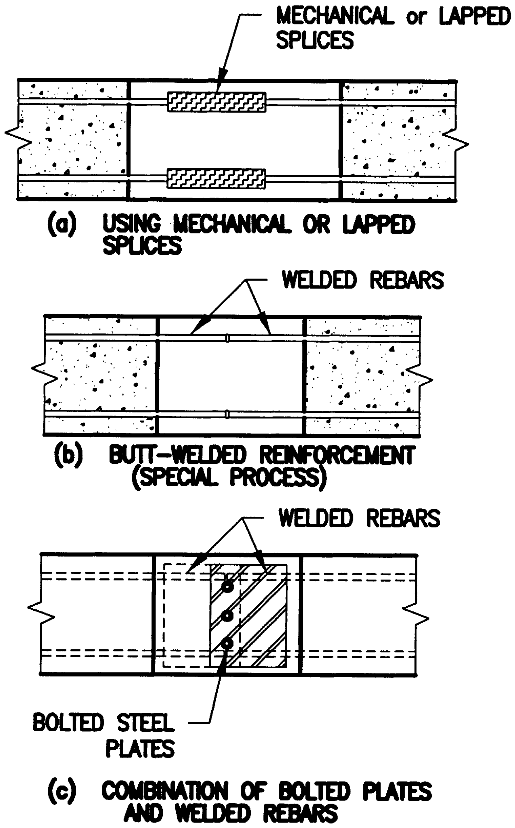

Fig. 16—Horizontal connections between beam or girder ends at locations other than column faces.

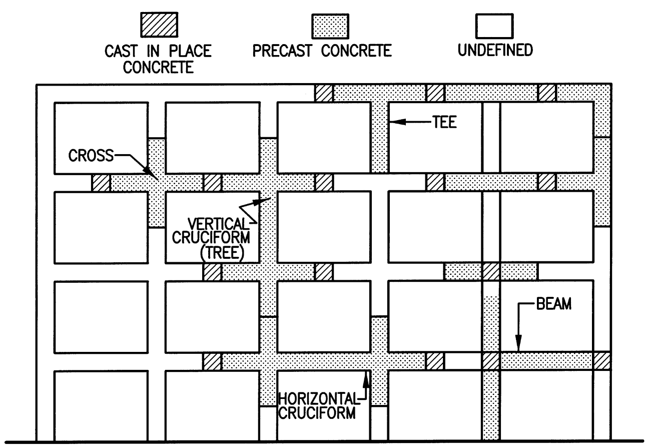

Fig. 17—Various configurations of precast frame elements.

both the strong connection region and the nonlinear action region. Noncontinuous reinforcement of the strong connection is to be developed between the connection and the beginning of the nonlinear action region. Lapped and welded splices are prohibited as connection hardware adjacent to a joint, the general area where the connection occurs;

3. The design strength of the strong connections is greater than a dynamic amplification factor Θ times the moment, shear, or axial force at the connection location based on the probable strength at the nonlinear action location. For column-to-column connections, Θ is 0.4. At these columns, transverse reinforcement for columns at joints is required

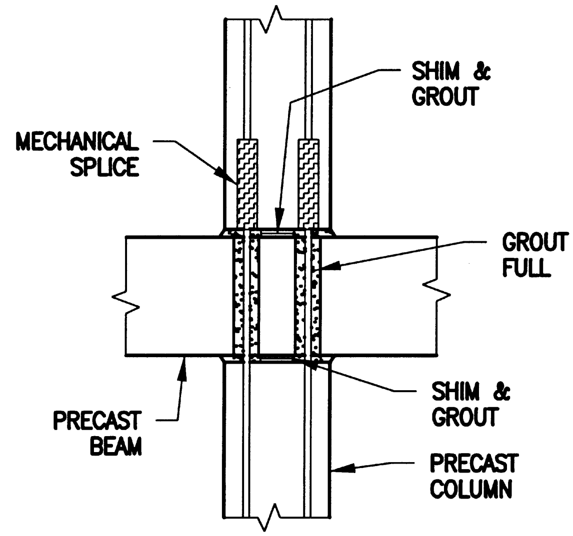

Fig. 18—Column-to-column connection through conduits installed in a beam.

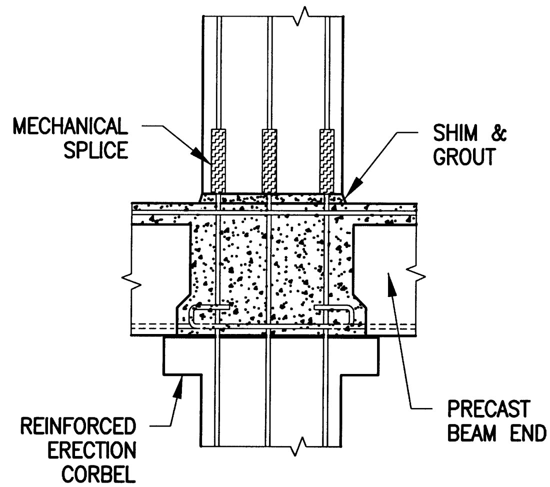

Fig. 19—Connection at beams and columns with a cast-in-place closure.

full-height. If the column-to-column splice is midheight, these requirements are subject to an exception that permits the moment strength of the connection to be 0.4 times the maximum probable flexural moment strength M_{pr} and the design shear strength to meet the requirements of ACI 318-99, Section 21.4.5.1; and

4. A strong connection located outside the middle half of a beam is to be a wet connection unless the dry connection can be substantiated by approved test results. A mechanical splice located within such a column-face strong connection (the connection at the surface or face of the column as opposed to being further back in the beam) is to be a Type 2 mechanical splice.

Other methods for seismic detailing of precast concrete are permitted by IBC 2000, but do not qualify under the definition of emulation.

4.3—Other connections—floor diaphragms

Satisfactory floor diaphragm connections are essential for obtaining acceptable diaphragm behavior and transferring

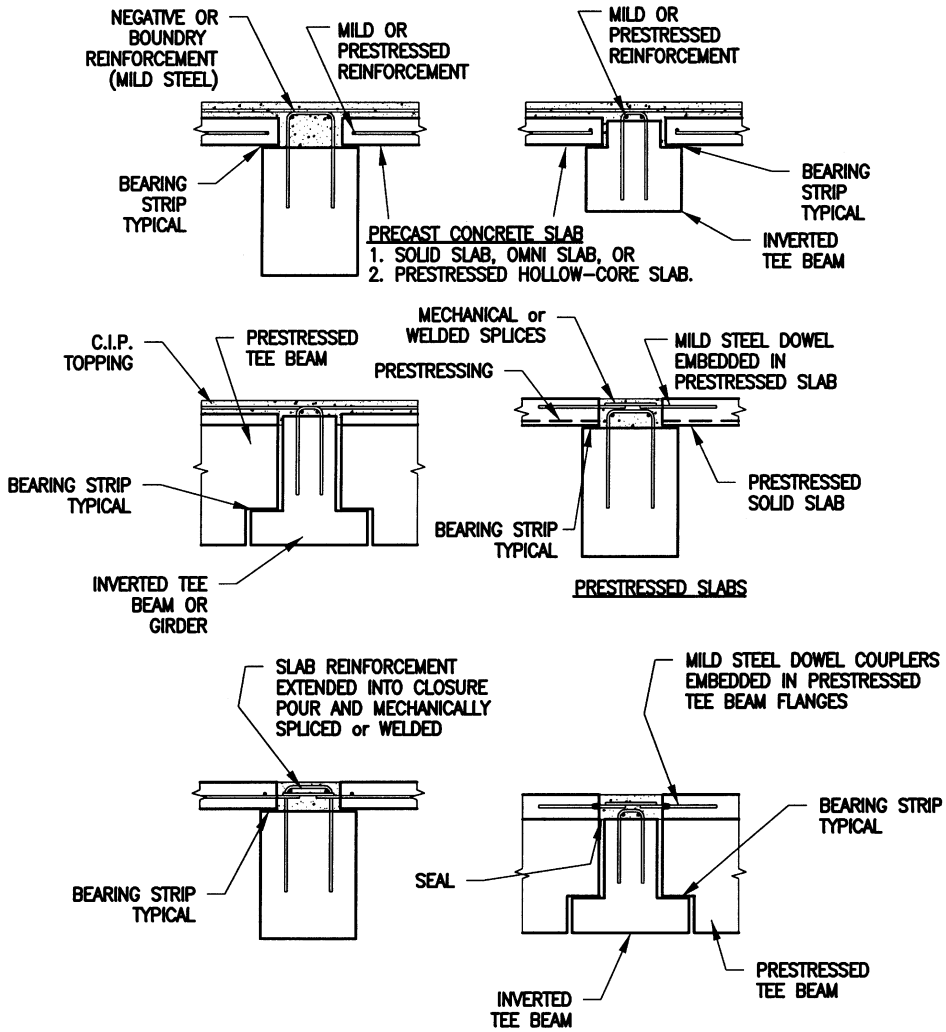

Fig. 20—Typical end connections of precast concrete floor slab elements.

the building's inertial forces to the lateral-load-resisting-system. A floor diaphragm can be a cast-in-place topping slab over precast floor elements or interconnected precast concrete floor elements. Figures 20 and 21 show a series of floor connections, between floor panels or between floors and supporting beams, that can be achieved by combining pour strips and spliced reinforcing bars.

Diaphragms using cast-in-place concrete topping are permitted by ACI 318-99 and as a modification to ACI 318-99, Chapter 21 made in Chapter 19 of the IBC 2000. The topping slab can be designed either as composite or noncomposite. Where mechanical splices are used to connect reinforcement between the diaphragm and the lateral system, the splice must develop 1.4 times the specified yield strength of the reinforcement. Currently, codes do not allow diaphragms composed of interconnected untopped precast elements in regions of high seismic risk.

The model building codes require the diaphragm to be designed for similar lateral forces as derived for vertical lateral-load-resisting elements. Ductility in the diaphragm is not preferred as it may result in the yielding and failure in the load path before the vertical elements experience the yielding forces or displacements they are intended to sustain.

Cast-in-place concrete diaphragms have added protection failure due to the uniform distribution of temperature steel in slabs (ACI 318-99, Section 7.12.2.1). With jointed diaphragms that are partially or totally formed with precast concrete, this inherent protection is not available. In the case of precast concrete systems, specific design considerations are needed to overcome the deficiencies in code provisions.

A detailed study of the behavior of precast concrete diaphragms has been reported (Nakaki 1998).

There has been a general tendency in reaction to the poor performance of some precast diaphragms in the Northridge earthquake to impose additional limitations on diaphragms in precast systems, including those on the diaphragm aspect ratio. This tendency may be misguided in an attempt to address the symptoms from poor design rather than to develop a rational protocol that ensures an effective system.

A significant difference between the cast-in-place slab and the precast floor with cast-in-place topping is the jointing. The jointing in the precast supporting the diaphragm slab tends to reflect as cracks in the cast-in-place. This discrete cracking can place a high strain demand on whatever reinforcing or connections cross these joints. ACI 318-99 has addressed this strain demand by setting a minimum spacing for wires in

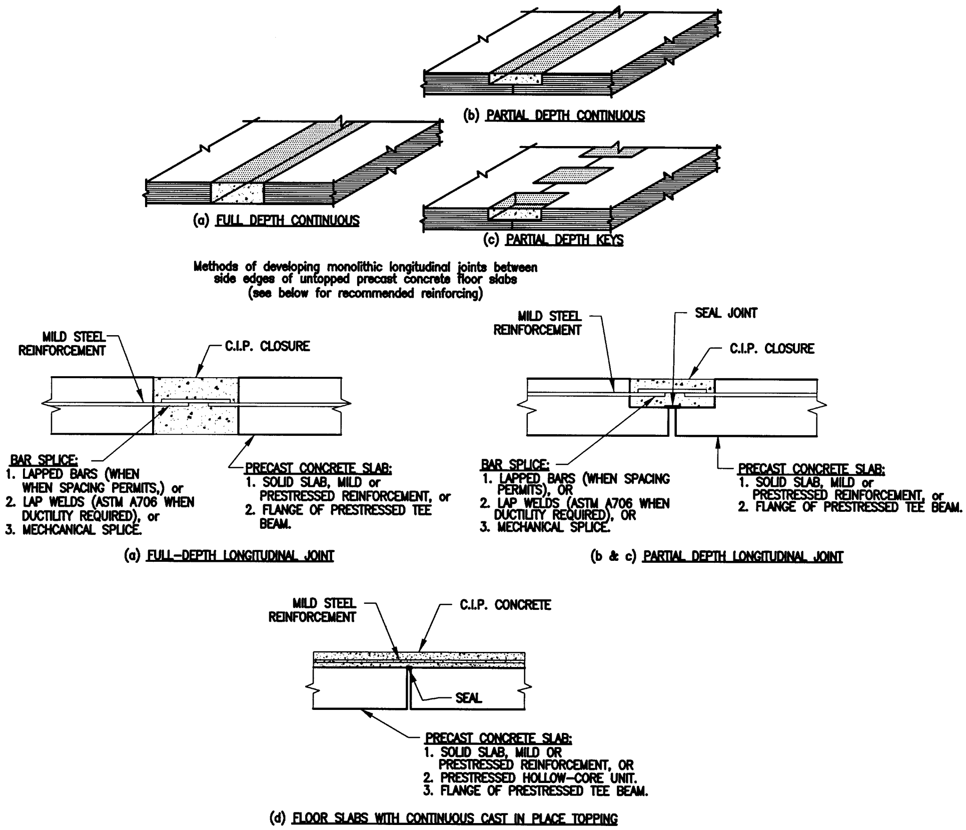

Fig. 21—Longitudinal joint between precast concrete partial thickness slabs with a cast-in-place topping.

welded-wire fabric in diaphragms of 10 in. (254 mm) for regions of high seismic risk. Similarly, mechanical connectors designed as part of the load transfer across joints should be capable of sustaining their design capacity under the concentrated strains that can accumulate at a joint. Connections intended for shear transfer only cannot be permitted to lose this capacity when the joint widens as an effect of flexure. The flexural (chord) reinforcing in the diaphragm should control diaphragm deformation not only to limit drift, but also to protect these elements from yielding.

Detailed design of precast diaphragms is beyond the scope of this report on emulation. A future report to address this need is under development within ACI Committee 550. Connections in box systems can be similar to wall and floor systems. In addition, where seismic conditions dictate a rigorous connection detail, those shown in Fig. 15 have been used. The overall concepts used in box systems are shown in Fig. 3 and 4. Details of the actual joint sections can be adopted as referenced on the diagrams.

4.4—Special materials and devices

In reinforced concrete, building codes allow splicing of reinforcing bars by means of lapping (except #14 and #18 bars), welding, and by use of mechanical splices. Neither welding nor lapping is permitted within potential plastic hinge regions. Reinforcing bars can be made continuous throughout the critical stress regions of precast concrete elements in much the same manner as they are for cast-in-place concrete and with the same restrictions as to type of splices permitted.

Structural ductility depends upon the inelastic (plastic) strain characteristics of the reinforcing bars and the concrete integrity within the plastic hinge. ASTM A 706 bars or equivalent should be specified when greater bar ductility is desired because the elongation capacity is approximately 50% greater than that of A615 steel. In Japan, reinforcing steels are used that have an elongation capacity about twice as high as ASTM A 706.

Figure 22 shows the generally available mechanical splices used in concrete construction. Some are readily adaptable for use in connecting precast concrete elements. Others are appropriate for splicing bars only in cast-in-place applications. Grout-filled splices are generally used with vertical reinforcing steel because they can be embedded completely inside the precast element without the need for an opening to access the splice during erection. Other types may be used in horizontal applications with cast-in-place closure placements. Because most splices are proprietary, the engineer should investigate the requirements and tolerances needed for a product under consideration as bars to be connected may be embedded and thereby impossible to turn or difficult to bend.

Most mechanical splicing devices are recognized by a model code body and may have formal conditions for acceptance in a structure. One of these conditions may be a requirement for special inspection.

To maintain the integrity of an emulative structure, grout specified as part of mechanical splicing devices should be mixed and installed according to the grout manufacturer's rec-

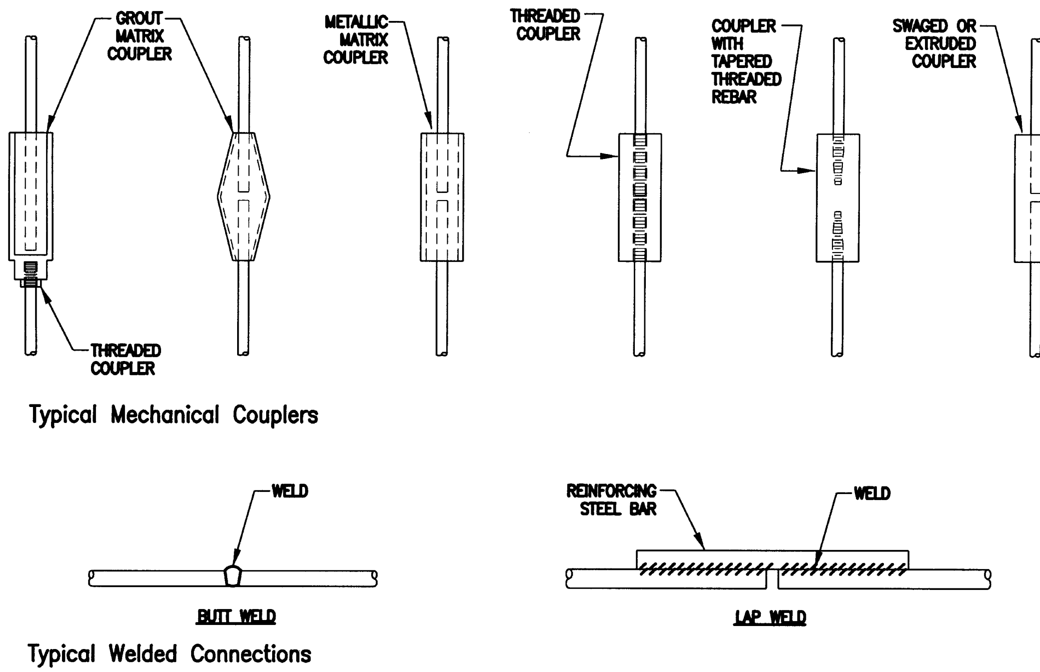

Fig. 22—Typical types of reinforcing bar splices.

ommendations. Grout or mortar used in sleeves, sheaths, conduit, bedding, and any other opening or void between or in the structural concrete elements should be carefully prepared and installed, with full attention paid to achieving the strength intended by the designer. The grout venting system should ensure complete placement throughout the connection.

Grouts or mortars used in the interfaces between precast concrete elements should be engineered. Grout strength should be specified and confirmed by the design engineer. Under no circumstances should interface grout be formulated or mixed at the job site by untrained persons or by using inappropriate equipment, such as a hoe and wheelbarrow, without proper means to measure and mix components without mixture proportion.

Grout field sample specimens should be made and cured according to ASTM C 109, C 942, or both, prescribed procedures, and tested by a recognized testing laboratory to ensure that the specimens meet specifications. For grout used in mechanical splices, a quality control program following the recommendations of the splice manufacturer is needed.

Conventional concrete mixtures can be used for closure placements to join precast concrete elements. The minimum strength of the concrete in closures should be the strength used in the precast elements.

For emulative purposes, most common connections include some form of reinforcement splicing, as reviewed previously. ACI 318-99, Type 2 mechanical splices need to be capable of sustaining a minimum of 100% of the specified ultimate strength of the rebar, which translates to 150% of specified yield strength. Research performed at the California Department of Transportation (Noureddine, Richards, and Grottkau 1996) shows that a minimum stress in the inelastic range in excess of 160% of the specified yield strength of the reinforcing steel is indicated to achieve 4% strain. Under the UBC-97, for highly active seismic regions, Type 2 mechanical splices in plastic hinging areas are required to develop at least 160% of the specified bar yield capacity.

CHAPTER 5—GUIDELINES FOR FABRICATION, TRANSPORTATION, ERECTION, AND INSPECTION

Fabrication of precast concrete elements for use in emulative precast concrete structures is little different than for most precast structural products. The primary difference is in the choice of connections for the reinforcing bars. To meet code requirements for cast-in-place concrete reinforcing bars should be made continuous and concentric through joints. To meet this requirement, bars may need to project through a member's end bulkheads. This may require modification of the bulkhead forms.

Transportation of emulative precast elements is similar to that for traditional precast concrete elements.

The type of connections used can speed erection. When erection is carefully planned for maximum efficiency, total crane time for a complete cycle of picking a wall panel, raising it, fixing it in place, and returning the slings back for the next element can be appreciably reduced. Precast cruciform elements were installed under optimum conditions at the upper stories of the 30-story MGM Grand Hotel in Las Vegas at the rate of 8 min. crane time per piece.

Inspection should focus on the connection system. Most ICBO evaluation reports for splices require that they be installed according to the manufacturer's instructions and under the special inspection requirement of the UBC. Design engineers will want to check that regional building code acceptance numbers have been issued for proprietary splicing devices.

The American Concrete Institute publishes numerous guidelines for quality control such as ACI 117-90. The Precast/Prestressed Concrete Institute (PCI) publishes a practical manual relating to erection practice (1999).

CHAPTER 6—EXAMPLES OF EMULATIVE PRECAST CONCRETE STRUCTURES

Many precast concrete structures using emulative technology have been constructed in the U.S. and Japan. Several significant examples are mentioned.

Because of its immense size and record-time assembly, the 30-story MGM Grand Hotel in Las Vegas, completed in 1994, is an interesting example of the use of emulative detailing. The exterior elements in the longitudinal frames were assembled from precast concrete frame cruciform members called trees. In the transverse direction, precast shear walls were used. The floors were precast, prestressed, untopped hollow core elements.

The 37-story Ohkawabata residential tower in Tokyo was constructed of precast concrete cruciform frames (Warnes 1990). Tokyo is located in one of the most severe seismic regions in the world. Simple beams and nonbearing partition walls between apartments were also fabricated of precast concrete. Balconies and floors were also constructed with half-thickness precast elements and cast-in-place floor topping. This method not only eliminates the need to erect and shore forms but also provides space for installing electrical conduit. More importantly, the topping ensures that a positive horizontal diaphragm is provided for each floor. Floor installation work proceeded directly behind the erection of frames, permitting follow-on trades to work on floors directly below the erection floor.

The standard design for FAA high-level (over 200 ft [61 m]) air traffic control towers use precast emulative detailing. Several of the towers are over 300 ft tall, including those at airports in Miami, Denver, and Dallas/Fort Worth. Recently, towers were built in Salt Lake City and Portland in high seismic zones. While they appear to be shear walls, the concept is actually a special moment-resisting frame to meet the 1997 Uniform Building Code restrictions (160 ft [49 m]) on the height of shear wall structures.

None of the cast-in-place concrete frame and shear wall structures constructed in the zone of influence of the 1995 Kobe earthquake, which were designed under the AIJ code provisions of 1971 and 1981, collapsed. Those built according to the 1981 AIJ code suffered only minor damage. A report (Architectural Institute of Japan 1996) on the performance of concrete structures during the Hyogoken-Nanbu (Kobe) Earthquake in 1995 illustrated the effect of improved code requirements. A significant number of cast-in-place reinforced concrete frame structures constructed under Japanese building code requirements in effect before 1971 collapsed or were severely damaged. A Japanese code change of 1971 consisted primarily of significantly increasing the amount of lateral reinforcement of columns by requiring additional column ties (hoops). None of the reinforced concrete building frame structures within the zone of strong motion influence of the Kobe event constructed under the provisions of the post 1971 code requirements collapsed, though there was severe damage to some.

A significant Japanese code change in 1981 introduced a code requirement for deliberately installing ductile hinges in beams at beam-column joints. This was done to ensure that plastic hinges would occur in beams at locations where they were desired. None of the post-1981 concrete frame buildings built under this code requirement, situated within the zone of influence of the strong seismic forces at Kobe, collapsed or experienced significant damage.

At Kobe, over 100 precast concrete box-frame (panel-type) structures located at 37 project sites within the zone of strong motion influence of the earthquake were not damaged and were approved for immediate occupancy after that seismic event (Ghosh 1995).

CHAPTER 7—SUMMARY AND CONCLUSIONS

All of the jointing details illustrated herein have been used in one form or another in the construction of emulative precast concrete structures. Many of these details, when experimental evidence of performance has been required by building officials as a condition for approval, have been tested for structural performance in laboratories in the U.S. and Japan. Building officials in Japan require that all proposed new jointing details be tested in a laboratory as a condition of approval. Some building officials in the U.S. also require testing verification of new joint details before approving their use.

Concrete structures using these connections can be designed according to contemporary standard reinforced concrete practice and current applicable building codes for reinforced concrete. Because the designs conform to the requirements of building codes for cast-in-place concrete, local building officials should recognize that emulative precast concrete structures meet the conditions of building codes for cast-in-place concrete and are not experimental.

CHAPTER 8—REFERENCES

8.1—Referenced standards and reports

The standards and reports listed as follows were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute (ACI International)

117 Standard Specifications for Tolerances for Concrete Construction and Materials

318 Building Code Requirements for Structural Concrete

American Society for Testing and Materials (ASTM)

A706/A706M Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement

C 109/C 109M Standard Test Method for compressive Strength of Hydraulic Cement Mortars (Using 2-in. [50-mm] Cube Specimen)

C 942 Standard Test Method for Compressive Strength of Grouts for Preplaced—Aggregate Concrete in the Laboratory

International Conference of Building Officials (ICBO)
Uniform Building Code™ 1997

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

American Society for Testing and Materials (ASTM)
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

International Conference of Building Officials
5360 Workman Mill Road
Whittier, CA 90601-2298

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