ACI T1.2-03

Special Hybrid Moment Frames Composed of Discretely Jointed Precast and Post-Tensioned Concrete Members

Reported by ACI Innovation Task Group 1 and Collaborators

Innovation Task Group 1

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This document defines requirements that may be used to design special hybrid moment frames composed of discretely jointed precast concrete beams post-tensioned to concrete columns. Such frames are suitable for use in regions of high seismicity or for structures assigned to satisfy high seismic performance or design categories. After a major seismic event, that moment frame can be expected to exhibit minimal damage in beam-column regions and negligible permanent displacements. Such moment frames do not satisfy the prescriptive requirements of Chapter 21 of ACI 318-99 for frames of monolithic construction. According to Section 21.2.1.5 of ACI 318, their acceptance requires demonstration by experimental evidence and analysis that the frames have strength and toughness equal to or exceeding those provided by comparable monolithic reinforced concrete frames that satisfy the prescriptive requirements of Chapter 21. This document describes the requirements that the designer may use to demonstrate, through analysis, that such frames have strength and toughness at least equal to those of comparable monolithic frames.

Among the subjects covered in this standard are requirements for:
1. Materials, and especially the special reinforcement that is debonded in the beam adjacent to the beam-column interface;
2. The framing system, and especially the roles of the post-tensioning tendons, the special reinforcement, and the floor slab; and
3. The beams of the moment frame, and especially their required prestress force level, the contribution of the special reinforcement to their probable moment strength, and the calculation of that strength.

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Keywords: drift ratio; energy dissipation; hybrid construction; lateral resistance; moment frame; post-tensioning; precast concrete; prestressed concrete; seismic design; test module; toughness.

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7.1—General

1.0—Introduction
1.1 Foreword—For regions of high seismicity, Section 21.2.1.5 of ACI 318 permits the use of structural systems that do not meet the prescriptive requirements of Chapter 21 if certain experimental evidence and analysis are provided. The intent of ACI T1.1-01/T1.1R-01, “Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary,” is to define the minimum evidence required when attempting to validate the use of weak-beam strong-column special moment frames in accordance with Section 21.2.1.5.

Before acceptance testing can be undertaken, ACI T1.1-01 requires that a design procedure be developed for prototype moment frames having the generic form for which acceptance is sought, and that procedure be used to proportion the test modules. ACI T1.2-03 defines the requirements to be used for one specific type of moment frame that does not satisfy the prescriptive requirements of Chapter 21 of ACI 318, but can be validated for use in regions of high seismicity under ACI T1.1-01. The moment frame uses precast concrete beams that are post-tensioned to precast or cast-in-place concrete columns. The columns are continuous through the joints, and the beams each span a single bay. This Standard describes the frame as a hybrid frame because it combines post-tensioned and precast concrete construction and because it combines the use of ordinary reinforcement that is designed to yield with unbonded post-tensioning tendons that are designed to remain elastic.

In this specific type of hybrid frame, the post-tensioning tendons are unbonded and designed to remain elastic during an earthquake. Horizontal reinforcing bars grouted in ducts located in the columns and in the top and bottom of the beams, and described in this document as special reinforcement, provide additional continuity between the beams and the columns, and additional moment strength to the beams. Those bars dissipate energy as they yield alternately in tension and compression during an earthquake.

A key feature of this system is that the grouted bars are deliberately debonded for a short distance in the beam adjacent to the beam-column interface in order to reduce the high cyclic strains that would otherwise occur at that location. Consequently, during an earthquake, the beams and columns displace essentially as rigid bodies with deformations occurring primarily at the beam-column interface as the beam rocks against the column.

A second key feature is that the post-tensioning allows the columns to be built without the permanent corbels normally found in precast concrete construction. The post-tensioning has two purposes. First, the friction induced by the post-tensioning transfers vertical shears at the interface between beam and column for both gravity and lateral loadings. Second, with the post-tensioning deliberately designed to remain elastic during a major seismic event, the post-tensioning forces the moment frame to return to its undeformed position following the event.

Under seismic loading, the special moment frames described in this document are intended to behave differently than monolithic frames. Most of the deformations of the frames occur from the opening and closing of the joint at the interface between the precast beam and the column. Consequently, with the detailing procedures described in this standard, damage during a major seismic event is limited in extent, confined essentially to the joint filler material, and can be readily repaired after the earthquake. By contrast, monolithic frames designed to Chapter 21 of ACI 318 can suffer significant cracking, crushing, and spalling in the plastic hinging regions of the beam, the beam-column joint, or both, and repair can be costly. Further, monolithic special moment frames designed to Chapter 21 of ACI 318 may show permanent lateral deformations following a major seismic event whereas the special moment frames described in this document should not.

The preceding five paragraphs define the key characteristics of hybrid frames. The detailing requirements described in this standard are for one specific type of special hybrid moment frame with:

(a) Equal moment strength for the top and bottom energy-dissipating reinforcing bars that cross the interface between the precast beam and column; and

(b) Post-tensioning tendons that are unbonded from anchor to anchor and concentrically located within the cross section of the beam.
Special moment frames with unequal moment strengths for the top and bottom energy-dissipating reinforcing bars, and with amounts, location, and bonding of the post-tensioning tendons that differ from those described in this document, can be proportioned to have performance characteristics similar to the frames described in this document. However, research investigations additional to those completed to date, and modifications of the requirements described in this document, are needed before prescriptive provisions for the design of such frames can be formulated.

Development of the frame described in this standard was made possible by the combined efforts of the National Institute for Standards and Technology (NIST); the University of California, San Diego; the University of Washington, Seattle; and the University of Illinois at Urbana-Champaign; the American Concrete Institute (ACI); the Precast/Prestressed Concrete Institute (PCI); numerous dedicated individuals; and the financial support of Charles Pankow Builders, Ltd., ACI’s Concrete Research and Education Foundation, and Dywidag Systems International.

1.2 Scope—

1.2.1—This document defines requirements for a certain type of special hybrid moment frame composed of precast concrete beams jointed at their connections to columns that are continuous past those joints. While these frames do not satisfy all of the prescriptive requirements of Sections 21.3 through 21.5 of ACI 318, analyses and tests and reporting requirements in accordance with ACI T1.1-01 have established dependable and predictable strength, energy dissipation, stiffness, and drift capacities for characteristic beam-column configurations of the frames described in this document.

1.2.2—The requirements described in this document are for special hybrid moment frames with:

1. Equal moment strength for the top and bottom energy-dissipating bars (special reinforcement) that cross the interface between the precast beam and column; and

2. Post-tensioning tendons that are unbonded from anchor to anchor and concentrically located within the cross section of the beam.

1.2.3—All precast and reinforced concrete components and systems for the moment frames, and the associated gravity load frames, shall be designed to satisfy the requirements of ACI 318 except as modified by this document.

1.2.4—The special inspection requirements of Section 1.3.5 of ACI 318 shall be satisfied for all precast and reinforced concrete components and systems for the special moment frames.

2.0—General

2.1 Notation—

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>( A_{vc} )</td>
<td>area of closed hoops or spirals within a distance ( s_{vc} ), in.²</td>
</tr>
<tr>
<td>( b )</td>
<td>width of compression face of precast beam, in.</td>
</tr>
<tr>
<td>( C )</td>
<td>compression force when probable flexural strength ( M_{pr} ) acts at interface, lb</td>
</tr>
<tr>
<td>( c )</td>
<td>distance from extreme compression fiber of grout pad to neutral axis at beam-column interface, in.</td>
</tr>
<tr>
<td>( d )</td>
<td>distance from extreme compression fiber of grout pad at interface to centroid of special tension reinforcement, in.</td>
</tr>
<tr>
<td>( d_p )</td>
<td>bar diameter of special reinforcement, in.</td>
</tr>
<tr>
<td>( E )</td>
<td>load effects of earthquakes, or related internal moments and forces</td>
</tr>
<tr>
<td>( f'_c )</td>
<td>specified compressive strength of concrete, psi</td>
</tr>
<tr>
<td>( f_{prs} )</td>
<td>stress in post-tensioning tendons when stress in special reinforcement is ( f_{pr} ), psi</td>
</tr>
<tr>
<td>( f_{pr} )</td>
<td>specified tensile strength of post-tensioning tendons, psi</td>
</tr>
<tr>
<td>( f_{py} )</td>
<td>specified yield strength of post-tensioning tendons, psi</td>
</tr>
<tr>
<td>( f_{se} )</td>
<td>effective stress in post-tensioning tendons (after allowance for all prestress losses), psi</td>
</tr>
<tr>
<td>( f_u )</td>
<td>specified tensile strength of special reinforcement crossing beam-column interface, psi</td>
</tr>
<tr>
<td>( f_{vcy} )</td>
<td>specified yield strength of closed hoops or spirals, psi</td>
</tr>
<tr>
<td>( f_y )</td>
<td>specified yield strength of special reinforcement, psi</td>
</tr>
<tr>
<td>( h )</td>
<td>overall thickness of the precast beam of a moment frame, in.</td>
</tr>
<tr>
<td>( h_p )</td>
<td>dimension of a column in the direction of the post-tensioning tendon, in.</td>
</tr>
<tr>
<td>( \ell_d )</td>
<td>development length in tension for a straight deformed reinforcing bar, in.</td>
</tr>
<tr>
<td>( L_{clear} )</td>
<td>clear span measured face-to-face of columns, in.</td>
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<tr>
<td>( L_d )</td>
<td>length over which special reinforcement crossing beam-column interface is deliberately debonded, in.</td>
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<tr>
<td>( L_{aps} )</td>
<td>length associated with a given interface over which post-tensioning tendon is unbonded, in.</td>
</tr>
<tr>
<td>( M_{pr} )</td>
<td>probable flexural strength at a beam-column interface of a precast concrete beam of moment frame, in.-lb</td>
</tr>
<tr>
<td>( M_{prs} )</td>
<td>contribution of post-tensioning reinforcement to ( M_{pr} ), in.-lb</td>
</tr>
<tr>
<td>( M_s )</td>
<td>contribution of special reinforcement to ( M_{pr} ), in.-lb</td>
</tr>
<tr>
<td>( N_{pr} )</td>
<td>effective post-tensioning force, ( A_{ps} f_{se} ), lb</td>
</tr>
<tr>
<td>( s_{vc} )</td>
<td>spacing of transverse reinforcement surrounding special reinforcement development length, in.</td>
</tr>
<tr>
<td>( V_c )</td>
<td>nominal shear strength provided by concrete, lb</td>
</tr>
<tr>
<td>( V_D )</td>
<td>shear force due to unfactored dead load, lb</td>
</tr>
<tr>
<td>( V_L )</td>
<td>shear force due to unfactored live load, lb</td>
</tr>
<tr>
<td>( V_n )</td>
<td>nominal shear strength, lb</td>
</tr>
<tr>
<td>( V_F )</td>
<td>factored shear force at section, lb</td>
</tr>
<tr>
<td>( \alpha_p )</td>
<td>coefficient quantifying the effective additional debonded length for special reinforcement at probable flexural strength</td>
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<tr>
<td>( \beta_l )</td>
<td>factor defined in Section 10.2.7.3 of ACI 318</td>
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<tr>
<td>( \Delta_{prs} )</td>
<td>elongation of post-tensioning tendon at probable flexural strength, in.</td>
</tr>
<tr>
<td>( \Delta_s )</td>
<td>elongation of special reinforcement at probable flexural strength, in.</td>
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318-99 Building Code Requirements for Structural Concrete

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

ASTM International
100 Barr Harbor Drive
West Conshohocken, Pa. 19428

International Code Council
5203 Leesburg Pike, Suite 708
Falls Church, Va. 22041-3401

2.2 Definitions—
The following definitions, additional to those of Section 21.1 of ACI 318 and of ACI T1.1-01, shall apply:

Nonlinear action location—Interface where end of precast beam of moment frame meets column face.

Reinforcement, ordinary—Reinforcement conforming to Section 3.5 of ACI 318, excluding prestressing tendons.

Reinforcement, prestressing—Post-tensioned tendons that cross the interface between precast beam and column and conform to Section 5.4.

Reinforcement, seismic—Reinforcement that conforms to Section 21.2.5 of ACI 318.

Reinforcement, special—Reinforcement (nonprestressed) that crosses the interface between the precast beam and the column, is debonded for a specified length in the beam adjacent to the beam-column interface and conforms to Section 5.3.

2.3 Drawings—
Drawings of the moment frames shall show all features of the work, including those details essential for satisfactory seismic performance of the frame. Essential details include: debonding the special reinforcement that crosses beam-column interfaces; anchoring the special reinforcement and tendons within beams and columns; and developing floor slab-frame interactions that conform to those assumed in the design documents.

2.4 Referenced standards—
American Concrete Institute (ACI)
318-99 Building Code Requirements for Structural Concrete
T1.1-01 Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary

ASTM International
A 370-96 Test Methods and Definitions for Mechanical Testing of Steel Products
A 416-96 Standard Specification for Steel Strand, Uncoated Seven Wire for Prestressed Concrete
A 706-96 Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
C 1107-97 Specification for Packaged Dry, Hydraulic Cement Grout (Nonshrink)

International Building Code (IBC)
IBC 2000 International Building Code

T1.2-4 ACI STANDARD

These publications may be obtained from these organizations:

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

ASTM International
100 Barr Harbor Drive
West Conshohocken, Pa. 19428

International Code Council
5203 Leesburg Pike, Suite 708
Falls Church, Va. 22041-3401

3.0—Materials
3.1 General—All materials and material tests shall conform to the requirements of ACI 318 except as specified in this standard.

3.2 Ducts—Ducts, including those for special reinforcement, shall conform to the requirements of ACI 318, Section 18.17.

3.3 Special reinforcement—
3.3.1—Special reinforcement shall have rib deformation heights, yield strength, and an ultimate elongation equal to or exceeding those required by ASTM A 706.

3.3.2—Unless specific test data on the stress-strain properties of the special reinforcement are obtained before design and construction, the stress-strain properties shall conform to ASTM A 706. For that latter case, the tensile strength shall be taken as the specified minimum tensile strength $f_{pu}$. The strain $\varepsilon_u$ at the tensile strength shall be taken as a strain 0.02 less than the strain at the minimum elongation specified in ASTM A 706 for the given bar size.

3.3.3—Where properties are based on test data as permitted by Section 3.3.2, the stress-strain properties of the special reinforcement for each bar size used in the moment frame shall be obtained from tension tests specified in ASTM A 706. The average strain $\varepsilon_p$ of that reinforcement at its average tensile strength $f_{pu}$ shall be obtained. Averages shall be based on the results of a minimum of three tension tests for each bar size for every steel heat used for the moment frame.

3.4 Prestressing strands and tendons—
3.4.1—In moment frames meeting the requirements of this document, post-tensioned prestressing strand tendons shall be used at nonlinear action locations, and pretensioned prestressing reinforcement shall be permitted in precast concrete flexural members.

3.4.2—Prestressing strand shall conform to ASTM A 416.

3.4.3—Prestressing strands shall be permitted to resist earthquake-induced forces, provided the strain in the strands does not exceed 0.011 at the limiting drift ratio of 0.035 specified in ACI T1.1-01.

3.4.4—Anchorages shall withstand, without failure, a minimum of 50 cycles of a loading for which the load in each cycle is varied between 40 and 80% of the minimum specified tensile strength of the prestressing strands of the post-tensioning tendon.
3.5 Interface grout—Nonshrink grout shall contain at least 0.1% fibers by volume and conform to ASTM C 1107.

3.6 Grout for anchorage of special reinforcement—Grout used to anchor the special reinforcement shall conform to the requirements of ACI 318, Chapter 18.

4.0—Framing system requirements
4.1 General—Designs shall provide:
    4.1.1—A continuous uninterrupted load path to the foundation for all components for dead load, live load, and wind and earthquake forces.
    4.1.2—Integrity of the entire load path when the structure, and every story in it, is subject to the limiting story drift ratio of 0.035.

4.2 Strength—At all sections, nominal strengths calculated in accordance with the requirements of ACI 318 and Section 6.0 of this document, multiplied by the strength reduction factors specified in ACI 318, shall equal or exceed the required strengths for all load combinations of Section 9.2 of ACI 318 involving the earthquake loading E.

4.3 Drift—
    4.3.1—The total drift ratio shall be computed as the lateral displacement at the top of the structure divided by its height.
    4.3.2—The story drift ratio shall be computed as the story drift divided by the story height.
    4.3.3—The maximum total drift and maximum story drift for the structure, of which the frame is part, shall be calculated as required by Section 1618 of the IBC 2000. Calculations shall include consideration of the soil type on the frame.
    4.3.4—The maximum total drift and story drift ratios, calculated as required in Section 4.3.3, shall not exceed 0.024.
    4.3.5—The drift ratio capacity for any joint shall be calculated as the sum of the components caused by: a) the inelastic deformations at the beam-column interfaces at the probable moment strengths for those interfaces; and b) the sum of the corresponding elastic deformations of the beams and columns framing into that joint and of the joint shear deformations.

4.3.6—The structure shall be designed to have maximum total drift ratio capacity and story drift ratio capacities, \( \theta_m \), equal to or greater than 0.035.

4.3.7—The story drift ratio capacity shall be the least drift ratio capacity for any joint in that story.

4.4 Moment frame characteristics—The precast concrete special moment frames described in this document shall, in addition to satisfying the requirements of Sections 21.2 through 21.5 of ACI 318, have characteristics meeting the requirements of Sections 4.4.1 through 4.4.5.

4.4.1—Single-bay precast concrete beams shall be used. Single and multistory precast concrete columns shall be permitted.

4.4.2—For the frames, the interfaces between the beams and the beam-column joints shall be the only nonlinear action locations, except for column-to-foundation connections.

4.4.3—For multistory precast columns, column lap-splice connections in any given story shall be permitted only within the middle third of the height between the top of the slab and the bottom of the beam, except at column-to-foundation connections. Type 2 connections, described in Section 21.2.6.1(a) of ACI 318, shall be permitted at any location within the column except within the beam-column joint.

4.4.4—Post-tensioning tendons in the beams shall be concentric and the force in the tendons shall:
    (a) have, as required by Section 6.2.1, an effective prestress \( f_{ps} \) that provides a clamping force across the beam-column interface sufficient to resist the shear caused by factored gravity loads; and
    (b) have a maximum stress \( f_{prs} \) at a story drift ratio of 0.035, as required by Section 6.5.5, that is less than the stress at a strain of 0.11 for the post-tensioning tendons.

4.4.5—The top and bottom special reinforcement in the beams shall have equal areas and equal strengths. This reinforcement shall be bonded through the column and debonded in the beam adjacent to the beam-column interface. This reinforcement shall have:
    (a) for both top and bottom bars, as required by Section 6.4.2, a strength that is both large enough to provide the relative energy dissipation ratio of not less than 1/8 required by ACI T1.1-01, and small enough that the effective prestress in the post-tensioning tendon can close any gap at the beam-column interface when earthquake motions cease; and
    (b) for the bottom special reinforcement, as required by Section 6.4.1, an area and yield strength sufficient to prevent collapse of the beam under unfactored gravity loads in the event of fracture of the post-tensioned reinforcement.

4.5 Distribution of moment frames within structures—In structures where moment frames are used in combination with precast-concrete gravity-load-carrying frames, the lateral force-resisting system shall be well distributed throughout the structure, as required by Section 1908.1.12 of the IBC 2000.

4.6 Moment frame-floor slab interactions—Moment frame-floor slab interactions shall satisfy the requirements of 4.6.1 and 4.6.2:
    4.6.1—The floor slab shall be designed and detailed, and its connections to the precast beams made, in such a manner that relative displacements at interfaces between beams and columns of hybrid frames can be consistent with the displacements anticipated at those interfaces based on the response characteristics established in the acceptance tests.
    4.6.2—The opening of the joints at the beam-column interfaces of the moment frames under seismic actions shall not affect the performance of either the gravity load system or the diaphragm.

5.0—Requirements for beams of moment frames
5.1 Prestress—Prestress effects shall conform to the requirements of ACI 318, Chapter 18, except that:
    (a) the provisions of Section 18.4 shall not apply for the load combinations required by Section 9.2 of ACI 318; and
    (b) the provisions of Section 18.9 of ACI 318 for minimum bonded reinforcement shall apply only in beam
regions outside of the regions where the special reinforcement is required to be debonded.

5.2 Beam design—

5.2.1—Shear strength of the beam for zero drift shall be computed using Eq. (11-2) of ACI 318 with $V_c$ computed by Eq. (11-4) of ACI 318 and $N_u$ taken as $A_{psfse}$.

5.2.2—The requirements of ACI 318 Section 21.3.4.2 for proportioning transverse reinforcement shall not apply to concrete in the beam adjacent to a beam-column interface.

5.2.3—At each location where the cross section of the beam changes, the shear strength shall be evaluated and adequate reinforcement shall be provided to resist the shears at those locations. Shear reinforcement shall be in the form of closed ties, welded wire fabric mats, or welded bar grids.

5.2.4—The ends of the precast beam shall be detailed to minimize the effects of crushing or spalling where concrete corners bear on the interface grout or the column. Those details shall be indicated on the drawings or in the project specifications.

5.2.5—The post-tensioning tendon anchorages and the special reinforcement, for the length over which it is debonded, shall be protected against corrosion. Details of the protection methods shall be indicated on the drawings or in project specifications.

6.0—Requirements for beam-column interfaces of moment frames

6.1 General—The interfaces at connections between beams of moment frames and columns shall satisfy the requirements of Sections 6.2 through 6.7.

6.2 Prestress force—

6.2.1—Minimum prestress force $A_{psfse}$ shall be

$$A_{psfse} = \frac{(1.4V_D + 1.7V_L)}{\phi \mu}$$

(6-1)

where $\mu$ is the coefficient of friction and equal to 0.6; and $\phi$ is the strength reduction factor for shear specified in Section 9.3.2.3 of ACI 318.

6.2.2—At the maximum probable flexural strength for the connection, the design vertical shear strength shall be equal to or greater than the required vertical shear strength. Unless it is demonstrated by test and analysis that an alternative procedure can be used, this requirement shall be satisfied as follows:

1. The required vertical shear strength shall be computed as specified in Section 21.3.4 of ACI 318; and

2. The design vertical shear strength $\phi V_n$ shall be taken as $\phi \mu C$, where $C$ is the compressive force resisted by the concrete at the interface. In computing $C$, the stress in the post-tensioning tendon shall be taken as $f_{prs}$, the stress in the special reinforcement in tension shall be computed in accordance with Section 6.5.4, and the stress in the special reinforcement in compression shall be taken as $1.25f_y$.

6.3 Interface grout—

6.3.1—The thickness of the nonshrink interface grout shall not exceed 1.5 in.
over which the post-tensioning tendon is unbonded. The stress $f_{pr}$ in the post-tensioning tendon at the probable flexural strength shall not exceed $f_{py}$.

6.5.6—Unless the stress in the special reinforcement in compression is calculated from deformation compatibility considerations for the compressed concrete and known stress-strain properties for the special reinforcement, the stress in that reinforcement at the probable flexural strength shall be assumed to be $1.25f_y$.

6.6 Anchorage of special reinforcement—

6.6.1—The special reinforcement shall be anchored by grout in ducts located in the concrete of the members on either side of the interface.

6.6.2—Development length $l_b$ for special reinforcement anchored in ducts shall be taken as $25d_b$ unless a lesser value is established from a set of tests.

6.6.3—Anchorages for the top and bottom special reinforcement crossing the interface shall be spliced to the matching top and bottom reinforcement of the precast beam. Closed hoops, spirals, welded wire fabric, or welded bar grids shall be used to confine the anchorage regions. The confinement reinforcement shall be provided both horizontally and vertically and shall extend the length of the anchorage. The confinement reinforcement shall have a yield strength $A_{vc}f_{vcy}$, computed as $0.7A_{ds}f_{ps}x_{vc}hl_b$ for the top reinforcement, and as $0.7A_{ds}f_{ps}x_{vc}h_l$ for the bottom reinforcement, where $x_{vc}$ is the spacing of the transverse reinforcement.

6.6.4—The restrictions on using splices in ACI 318, Section 21.3.2.3, shall not apply where load transfer from the special reinforcement to the longitudinal reinforcement of the precast beam bars conforms to Section 6.6.3.

6.7 Distribution of flexural reinforcement—The provisions of ACI 318, Section 10.6, on distribution of flexural reinforcement shall not apply to the special reinforcement.

7.0—Frame joints

7.1 General—

7.1.1—Frame joints shall be designed in accordance with the requirements of Section 21.5 of ACI 318. Nominal shear strengths shall be calculated using effective joint areas $A_j$, for which effective widths include deductions for the width of the post-tensioning ducts.

7.1.2—The design shear strength of the joint shall not be taken as greater than $\phi$ times the value specified in Section 21.5.3.1 of ACI 318. A lesser value shall be used if the appearance of that joint following a major seismic event is of concern.
Commentary on Special Hybrid Moment Frames Composed of Discretely Jointed Precast and Post-Tensioned Concrete Members

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R1.0—Introduction

R1.2 Scope—Laboratory studies\(^1\)\(^-\)8 have shown that precast or prestressed concrete moment frames can provide safety and serviceability levels, during and after an earthquake, that meet or exceed performance levels required by Section 21.2.1.5 of ACI 318.\(^9\) To achieve such performance levels, the precast or prestressed concrete moment frames should be carefully proportioned and detailed. This document is based on the studies reported in References 1 to 4 and 6 to 8, and their references. It contains the minimum requirements for ensuring that one type of precast and prestressed concrete moment frame can sustain a series of oscillations into the inelastic range of response without critical decay in strength or excessive story drifts. Further, that frame should show only minimal or no damage in beam-column joint regions and no permanent displacements after the oscillations cease.

The type of structure covered by this document is illustrated in Fig. R1.2. That figure shows details for a typical interior lateral-load-resisting frame in Fig. R1.2(a), details for a typical interior beam-column joint in Fig. R1.2(b), and cross sections through the column on the axis of the beam and through the beam at a section containing special reinforcement in Fig. R1.2(c). The frame is composed of multistory columns to which single-bay precast concrete beams are connected. Except for possible yielding at the column bases, the interfaces between the precast beams and the continuous columns are the only locations where yielding of the reinforcement (nonlinear action location) occurs in the frame during a major seismic event. Crossing each interface are three deliberately debonded reinforcing elements: post-tensioned strands that extend the full length of the frame in the direction of its plane; and top and bottom deformed bar special reinforcement that is anchored by grouting in ducts preformed in the beam and column. The length over which the special reinforcement is debonded in the beam adjacent to the connection is selected deliberately to provide the desired design level of overall performance. Reference 10 describes the development of a rational basis for the design procedures for a frame with equal strength for the top and bottom debonded special reinforcement and with central post-tensioning tendons that remain elastic during a major seismic event.

R1.2.3—For the hybrid moment frames described in this document to be accepted as special moment frames, the special detailing of the frames needs to be properly executed through continuous inspection by personnel who are properly qualified to do that work.

R2.0—General

R2.1 Notation—While areas of the top and bottom special reinforcement are designated as \(A_s\) and \(A_s'\), respectively, those two areas must be equal for the special hybrid moment frames described in this document. Different symbols are used for the two steels to facilitate discussions and not to imply that the two areas may differ.

R2.2 Definitions—The term “nonlinear action location” is introduced from NEHRP\(^11\) and it prompts the designer to recognize differences between the behavior of hybrid moment frames formed from discretely jointed precast members and moment frames of monolithic construction. For strong-column, weak-beam monolithic construction, inelastic rotations that occur where the beam intersects the column are distributed over a length of the beam approximately equal to its depth. The center of the nonlinear action location is at the center of that length. For the same construction with hybrid moment frames composed of discretely jointed precast members, designed in accordance with this document, the inelastic rotations are concentrated at the precast beam-

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**Fig. R1.2**—Typical moment frame composed from discretely jointed precast concrete members: (a) elevation of typical interior moment frame; (b) detail of connection—A; and (c) typical sections—B and C.
column interface. That interface is the center of the nonlinear action location.

The definition of reinforcement in ACI 318\(^9\) does not permit easy differentiation between the four types of reinforcement permitted in special moment frames designed in accordance with this document. The designer can choose ordinary reinforcement for the precast beams because this document requires that those beams be designed to remain elastic when moments \(M_{pr}\) act on their ends. By contrast, reinforcement in the column may be locally stressed inelastically when the beam develops \(M_{pr}\). The designer then needs to use seismic reinforcement conforming to Section 21.2.5 of ACI 318\(^9\) for the column. The reinforcement, other than the post-tensioning tendons, crossing the beam-column interface can have properties deliberately chosen to differ from those for the column and beam reinforcement. Therefore, the term “special reinforcement” is used to describe those bars.

**R2.3 Drawings**—Because reinforcing details in the region where the precast concrete beam is connected to the continuous column are essential to the satisfactory performance of the moment frame in a major seismic event, the details should be designed meticulously and fully documented on the drawings for each connection of the moment frame. For the special reinforcement, essential elements include the length that reinforcement is to be deliberately debonded, how it is to be debonded, and details of its anchorage in the adjacent precast beam and column. For the post-tensioning tendon, essential elements are how and where the tendon will be anchored, and how it will be ensured that the tendon remains debonded during the grouting of the joint between the precast beam and column. Further, because satisfactory performance of the hybrid moment frame requires that yielding be limited to the special reinforcement crossing the beam-column interfaces, floor slab details in that connection region should be planned and executed so that they do not adversely affect the effective strength or stiffness of the connection.

### R3.0—Materials

**R3.3 Special reinforcement**—Stress-strain properties of the special reinforcement need to be defined accurately. The strength of the beam-column connection and the displacement of the frame are controlled by the maximum strain developed in that special reinforcement and its effective debonded length. The maximum strain demand placed on that reinforcement reaching its tensile strength \(f_p\), where \(\varepsilon_u\) and \(f_p\) have the meanings shown in Fig. R3.3. If ASTM A 706\(^{12}\) steel is specified for the special reinforcement and no testing has been performed specific to the reinforcement used in the frame, then the designer should use limiting strain values that are less than the minimum elongations specified for that steel. It is important to note that \(\varepsilon_u\) is less than the minimum elongation \(\varepsilon_f\) in Fig. R3.3, specified for the corresponding bar size in ASTM A 706. The strain increase between \(\varepsilon_u\) and \(\varepsilon_f\) is due to local necking of the bar. That difference in strain increases as the bar size decreases. In the absence of specific test data for the difference between \(\varepsilon_u\) and \(\varepsilon_f\), that value can be taken as 0.02 (2%) for ASTM A 706 steels.

![Typical stress-strain relationship for special reinforcement](image)

**R3.4.1**—Using prestressing reinforcement in special moment frames in regions of high seismic risk is not specifically allowed currently by Section 21.2.5 of ACI 318\(^9\). Using pretensioned prestressing tendons as the reinforcement in the precast beam is permitted by this document because those beams are constrained to remain within the elastic range of response, except at the beam-column interfaces. Using unbonded post-tensioned tendons consisting of prestressing strands in the beams is also permitted by this document. The use of bars instead of strands at nonlinear action locations is not permitted due to the significant additional stresses at high displacements caused by kinking of the bars where they cross the beam-column interfaces.

**R3.4.3**—Normally, it is considered good practice during construction to jack prestressing tendons to the highest force consistent with not causing permanent deformations. For the strands crossing the beam-column interface, the maximum permissible jacking stress will often be constrained by the design and may be as low as 0.4f\(_{pu}\) to ensure that the strand does not yield in a major event.

However, because the prestress force is also to prevent sliding due to vertical shear forces at the beam-column interface, the accurate calculation of prestress losses becomes more essential when low prestress levels are used.

### R4.0—Framing system requirements

The design of the frame is controlled primarily by drift, rather than strength, considerations. Performance requirements for the moment frame, the post-tensioning tendons, and the special reinforcement for any moment frame composed from discretely jointed precast concrete members connected by a combination of post-tensioning and special reinforcement are given in Section 4.1. Specific detailing requirements for a frame with concentric post-tensioning and equal top and bottom special reinforcement are given in Sections 5.1 and 6.1.

**R4.1 General**—The integrity of the load path to the foundation for all components should be examined for the position to which the structure deforms at a maximum anticipated drift ratio of 0.035. This drift ratio requirement can be satisfied by examining the integrity of the load path when each story is deformed to the limiting drift ratio of 0.035.

**R4.3 Drift**—The moment frame has to satisfy both total and story drift constraints. Generally, the latter control the design.
R4.4 Moment frame characteristics—The minimum structural characteristics required of the moment frame, regardless of the details of its design, are specified in Section 4.5.

R4.4.2—For strong-column, weak-beam construction, the column overstrength factor $\lambda$ equal to the ratio of the nominal flexural strengths of the columns framing into a given beam-column joint, $\Sigma M_c$, to the nominal flexural strengths of the beams framing into the same joint, $\Sigma M_g$, needs, as specified in ACI 318\textsuperscript{9} Section 21.4.2.2, to be equal to or greater than 1.2. For this document, values for $\Sigma M_c$ and $\Sigma M_g$ are to be calculated as specified in ACI 318.\textsuperscript{9} The designer should, however, choose a value for $\lambda$ consistent with the performance of joints established in the acceptance tests.\textsuperscript{13} That value should be not less than 1.2 increased by factors that account for differences in column axial forces, slab reinforcement, etc., anticipated for the prototype building and not present in the modules for which testing is reported in References 7 and 8.

R4.4.3—Columns may be precast or cast-in-place, provided they are continuous through beam-column joints.  

R4.4.4—It is necessary that the post-tensioning tendons continuous through the interior column joints and the beams of the moment frame satisfy the two conditions specified in Section 4.4.4. There should be no slip of the precast beam relative to the column, either under gravity loads or under the maximum forces in a seismic event. To ensure the preceding condition and that the frame will not show permanent displacements following a major event, the post-tensioning tendons passing through the beam-column interface must remain elastic throughout that event. Further, the effective prestress force in the post-tensioning tendons needs to be sufficient to cause compressive yielding in the top and bottom special reinforcement. Only then will any gap between the beam and the column that develops during an event close after oscillations cease, because the special reinforcement develops permanent elongations as those bars yield in tension.

R4.4.5—The special reinforcement crossing the interface fulfills two functions. First, it is the primary source of energy dissipation for the frame during a seismic event. For that function, the reinforcement needs, as required by T1.1-01,\textsuperscript{13} to provide a relative energy dissipation ratio exceeding 1/8. Second, the special reinforcement acts as integrity steel additional to that provided by the post-tensioning tendons. For that function, the special reinforcement is anchored in the column and the beam, and is designed to support the gravity loads acting on the precast beam in the unlikely event that a post-tensioning tendon fractures during an earthquake or due to some other cause.

R4.5 Distribution of moment frames within structures—Damage to precast concrete structures in recent earthquakes raised concerns about whether ACI 318-95 provisions were adequate to enable the inertial forces acting on structures with precast gravity load frames to be transmitted by the diaphragms to the lateral-force-resisting elements. IBC 2000,\textsuperscript{14} Section 1908.1.12, contains provisions intended to provide improved performance of structures having precast concrete gravity-load-carrying systems. Those provisions should be satisfied for buildings containing moment frames conforming to this document. Using the second method governing design of the beam-to-column connections, rather than the first method specified in those provisions, is recommended.

R4.6 Moment frame-floor slab interaction—Special attention should be given to the detailing of the connection of the floor slab to the column, to the precast beams, and to any gravity load beam framing into the same column of a given intersection in the moment frame. The presence of the floor slabs, or the gravity loads acting on the precast beams, can change the elastic behavior of the post-tensioning tendons and the flexural strength of the columns. Additional analysis and acceptance testing are necessary, however, to establish the performance for the frame consistent with Sections 4.1 through 4.4. The requirements call for equal amounts of top and bottom special reinforcement and concentrically located post-tensioning tendons crossing the beam-column interfaces. To date, performance characteristics have been validated by tests\textsuperscript{7,8} only for interfaces with those properties.

R5.0—Requirements for beams of moment frames  

In Sections 5.0 and 6.0, requirements are given for the precast beams and the beam-column connections of the moment frame. Those requirements are intended to provide a performance for the frame consistent with Sections 4.1 through 4.4. The requirements call for equal amounts of top and bottom special reinforcement and concentrically located post-tensioning tendons crossing the beam-column interfaces. To date, performance characteristics have been validated by tests\textsuperscript{7,8} only for interfaces with those properties.

Connections with nonconcentric post-tensioning, as are likely when part of the gravity loads acting on the precast beam are balanced by post-tensioning tendon forces, or connections with unequal top and bottom reinforcement, are likely when the post-tensioning is nonconcentric, can be designed to perform satisfactorily. Additional analysis and acceptance testing are necessary, however, to establish design procedures to augment those of Sections 5.0 and 6.0.

R5.1 Prestress—The post-tensioning tendons fulfill three functions. They provide a reliable clamping force to resist shears caused by gravity loads and seismic forces. They
provide moment resistance at the beam-column interface additional to that provided by the special reinforcement. Finally, by remaining elastic during a major event, they can close any gap at the beam-column interface when motions cease and therefore return the frame to its initial undeformed condition. Those considerations relate primarily to strength, rather than serviceability, so that stresses at service loads are generally not a concern.

The strength of the special reinforcement crossing the interface, in relation to the strength of the post-tensioning tendons crossing that same interface, is selected to provide certain predetermined response characteristics. Applying the minimum bonded reinforcement provisions of Section 18.9 of ACI 318(9) to the interface region of the precast beam is inappropriate. In the central region of the precast beam, however, outside the area where the special reinforcement is anchored, those minimum reinforcement provisions apply.

R5.2 Beam design—

R5.2.1—In the end region of the precast beam, the amount of stirrup reinforcement required depends on the contribution of the shear strength of the concrete to the total shear strength of the member. Because the special reinforcement is partly debonded in that region, the cracking pattern that develops differs from that expected for a reinforced concrete member with bonded steel. Conversely, if the section is treated as prestressed, it is not immediately clear which of the shear strength formulas of ACI 318(9) is appropriate. Correlating the performance of the end regions of test beams with ACI 318(9) expressions suggests that an appropriate approach is to consider the prestress force as an axial compression force acting on the gross area of the beam at the interface.

R5.2.2—The cross section of the precast beam changes at locations where an allowance is made for the insertion of the special reinforcement. Shear failures may occur at such locations if inclined cracking develops and there is inadequate shear reinforcement.

R5.2.3—Under severe displacement cycles, the corners of the precast beam are likely to crush or spall unless preventative measures are taken. Reinforcement spirals surrounding the ducts that contain the special reinforcement are desirable to prevent any loss of moment capacity with crushing or spalling. In addition, there are at least two other possible approaches that can assist in minimizing the effects of crushing. Steel angles or other reinforcement, such as carbon-fiber or aramid sheets, can be used to confine the beam corners. The reinforcement should extend from the corner of the precast beam to the depth of the special reinforcement in the direction of the thickness of the beam and from the corner to at least twice that depth in the direction of the span of the beam. The reinforcement should have sufficient thickness and anchorage adequate to provide the required confinement at the strain levels anticipated in a major event. Alternately, the beam corners can be chamfered and the presence of that chamfering considered in the design. The geometry for the grout pad created between the beam and the column needs to be consistent with the geometry for the reinforcement or chamfering used on the end of the beam.

R6.0—Requirements for beam-column interfaces of moment frames

R6.2 Prestress force—

R6.2.1—Because shear failure occurs through the grout in the pad between the beam and column, the coefficient of friction is taken as 0.6. When the post-tensioning tendon remains elastic under the design drift demand, use of Eq. (6-1) permits the performance requirement of Section 4.4.4(a) to be met.

R6.2.2—Under seismic action, the shear demand at the beam-column interface is a function of both the gravity loads acting on the precast beam and the seismic moments induced in it. In accordance with Fig. R21.3.4 of ACI 318(2), the design shear force \( V_u \) is given by:

\[
V_u = 0.75(1.4 V_D + 1.7 V_L) + \frac{(M_{pr1} + M_{pr2})}{L_{clear}} \quad (R6-1)
\]

where \( M_{pr1} \) and \( M_{pr2} \) are the values of \( M_{pr} \) for opposite ends of the deforming precast beam, and \( L_{clear} \) is the face-to-face distance between columns.

When the coefficient of friction is 1.0, the nominal shear strength equals the compression force acting at the interface

\[
\phi V_n = \phi C
\]

where \( C = A_s f_u - A_s' 1.25 f_y + A_{ps} f_{prs} \).

R6.3 Interface grout—The performance of the joint between the beam and the column directly depends on the toughness of the grout used in the interface. The interface is also the only location where erection tolerances can be provided. The grout in the joint should remain intact and not crush or fall out before the end of the precast beam starts spalling in an extreme event. Using fiber reinforcement in the grout is desirable to ensure adequate toughness. Fibers may be steel or polypropylene. Polypropylene fibers are desirable in exposed locations because joints with steel fibers can be susceptible to rusting.

If the joint is too wide, the grout can fail under a combination of shear and axial stress at stresses less than the compressive strength of the grout. The compressive strength of the grout should be approximately the same as the compressive strength of the precast beam. If the grout has a strength considerably greater than that of the beam, it can cause premature crushing of the concrete in the end of the beam at high strain levels. If the grout has a strength considerably less than that of the beam, it will crush first at high strain levels and cause a prestress loss, the effects of which will be difficult to offset during any subsequent repair operations.

R6.4 Special reinforcement—

R6.4.1—Where the special reinforcement is properly anchored in both the column and the precast beam, and the width of the joint at the interface is relatively small, the shearing yield stress of a bar is approximately half its tensile yield stress. The performance requirement of Section 4.4.5(b) can be met by satisfying the condition...
The probable flexural strength $M_{pr}$ is the sum of the contributions from the special reinforcement $M_s$ and the post-tensioned reinforcement $M_{prs}$. Those moments are given by

$$M_s = A_s f_{yd} \left( d - \frac{\beta_1 c}{2} \right) - A_s' 1.25 f_y \left( d' - \frac{\beta_1 c}{2} \right) \quad (R6-6)$$

when the stress in compression in the special reinforcement is taken as $1.25 f_y$ and

$$M_{prs} = A_{prs} f_{prs} \left( h - \frac{\beta_1 c}{2} \right) \quad (R6-7)$$

where

$$M_{pr} = M_s + M_{prs} \quad (R6-8)$$

and $f_{prs}$ is the stress corresponding to $\epsilon_{prs}$.

For a design earthquake having a 10% probability of occurrence in 50 years, the design drift concept should be consistent with the design displacement concept specified in Section 21.1 of ACI 318. As discussed in T1.1-01, the target design drift should be considerably less than the drift capacity of 3.5%. A target design drift of approximately 2% is desirable for a design earthquake with a 10% probability of occurrence in 50 years.

**R6.5.3**—It is desirable that the designer examine variations in the calculated response with variations in values for $L_u$ and $\alpha_b$ before settling on a design detail for the deliberately debonded region of the special reinforcement. As $\alpha_b$ values vary from 2.0 to 5.5, values for $M_{pr}$ increase slowly while values for drift increase more rapidly.

**R6.6 Anchorage of special reinforcement**

**R6.6.2**—Bars anchored in concrete confined by metal ducts, such as the spirally wound light gage steel ducts used for grouted post-tensioned construction, require less development length than bars anchored in monolithic concrete. For a Grade 60 (420 MPa) steel and a 5000 psi (35 MPa) concrete, Section 21.5.4 of ACI 318 requires a 32.6 development length for a straight bar. Because of the presence of the metal duct and the large amount of transverse reinforcement required by 6.6.3, a shorter development length can be required for the special reinforcement than that specified in Section 21.5.4 of ACI 318. Analysis of available results shows that a length of 25d_b is appropriate if no fibers are used in the grout, and even shorter lengths are appropriate if fiber is used in the grout. In the end column of a given frame, the column thickness $h_p$ should be large enough to anchor the special reinforcement. For the interior column of a multi-bay frame, special reinforcement should pass through the column and be anchored in both precast
beams as well as the column. In a major earthquake, the special reinforcement passing through an interior column is subject to a compressive force at one face of the column and a tensile force at the other face. The minimum column thickness should exceed the minimum development length for the bar.

**R6.6.3—**These confinement requirements have been derived from the results of tests on lap splices used at the base of bridge columns.\(^1\)\(^2\)\(^3\) For fully reversed cyclic loads, those tests showed that, with adequate confinement, stresses in splices with development lengths as small as \(25d_b\) could reach the tensile strength of the bar. That condition was achieved even though all the longitudinal bars of the column were spliced at the base to the same number of dowel bars protruding out of the foundation. The confining reinforcement had to be adequate to prevent sliding on a potential splitting plane separating the column and dowel bars. The situation at the end of the precast beam is analogous to that at the base of a bridge column. The special reinforcement needs to be anchored in the precast beam. The transverse reinforcement needs to be adequate to splice the special reinforcement to the longitudinal reinforcement of the precast beam. Figure R6.6.3 shows the worst potential splitting plane for a precast beam with a cross section the same as that of the precast beams of Reference 21. In those beams, the special reinforcement was placed in a trough and lap-spliced to the longitudinal reinforcement.\(^7\) Further, because the intention is to limit damage during a major earthquake to the joint filler material, conservative procedures should be used to compute joint shear strengths. The effective area of the joint is reduced by the presence of the post-tensioning duct. For the joint of an exterior column, there can be an offsetting effect resulting from the anchorage of the post-tensioning duct at the face of the column remote from the beam. That anchorage provides a reaction for the compression strut that forms within the beam-column joint under lateral loads. The existence of that reaction reduces the vertical projection of that strut to considerably less than the full depth of the beam-column joint and therefore reduces confinement requirements for the joint. For the joint of an interior column, however, there is no such offsetting effect, and the reduction in effective joint area \(A_p\) caused by the presence of the post-tensioning duct, is significant.

**APPENDIX—CITED REFERENCES**


15. Chagnon, M., “Precast Seismic Resisting Frames Using Unbonded Prestressing Tendons,” Report to the Precast/Prestressed Concrete Institute on Research in Progress at the University of California, San Diego, private communication, 1998.


