STANDARD

COMMENTARY

CHAPTER 6—CONCRETE FRAMES WITH INFILLS

6.1—Types of concrete frames with infills

Concrete frames with infills consist of complete gravity-load-carrying concrete frames infilled with masonry or concrete, constructed in such a way that the infill and the concrete frame interact when subjected to gravity and seismic forces.

Infills are considered to be isolated from the surrounding frame when the minimum gap requirements specified in 11.4.1 of ASCE 41-17 are satisfied. If all infills in a frame are isolated, the frame shall be analyzed as an isolated frame according to provisions given in Chapters 6, 7, and 11, and the isolated infill panels shall be analyzed according to the requirements of Chapter 11 of ASCE 41-17.

6.1.1 *Types of frames*—The provisions of Chapter 6 shall apply to concrete frames, as defined in Chapters 4, 5, and 9, which interact with infills.

6.1.2 *Masonry infills*—The provisions of Chapter 4 shall apply to masonry infills, as defined in Chapter 11 of ASCE 41-17, which interact with concrete frames.

6.1.3 *Concrete infills*—The provisions of Chapter 6 shall apply to concrete infills that interact with concrete frames, where the infills were constructed to fill the space within the bay of a complete gravity frame without special provision for continuity from story to story. The concrete of the infill shall be evaluated separately from the concrete of the frame.

C6.1—Types of concrete frames with infills

C6.1.3 *Concrete infills*—The construction of concrete-infilled frames is similar to that of masonry-infilled frames, except that the infill is of concrete instead of masonry units. In older existing buildings, concrete infill commonly contains nominal reinforcement, which often does not extend into the surrounding frame elements. The concrete used in the infill is often lower quality than that used in the frame elements and should be evaluated separately from investigations of the frame concrete.

6.2—Concrete frames with masonry infills

6.2.1 *General*—The analytical model for a concrete frame with masonry infills shall represent strength, stiffness, and deformation capacity of beams, slabs, columns, beam-column joints, masonry infills, and all connections and components of the element. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with nonstructural components shall be included.

For a concrete frame with masonry infill resisting seismic forces within its plane, modeling of the response using a linear elastic model shall be permitted, provided that the infill does not crack when subjected to design seismic forces. If the infill does not crack when subjected to design seismic forces, modeling the assemblage of frame and infill as a homogeneous medium shall be permitted.

For a concrete frame with masonry infills that cracks when subjected to design seismic forces, modeling of the response using a diagonally braced frame model, in which the columns act as vertical chords, the beams act as horizontal ties, and the infill acts as an equivalent compression strut, shall be permitted. Requirements for the equivalent

C6.2—Concrete frames with masonry infills

C6.2.1 *General*—The licensed design professional is referred to FEMA 274 and FEMA 306 for additional information regarding the behavior of masonry infills.

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STANDARD

COMMENTARY

compression strut analogy shall be as specified in Chapter 11 of ASCE 41-17.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill, as specified in Chapter 11 of ASCE 41-17. In frames with full-height masonry infills, the evaluation shall include the effect of strut compression forces applied to the column and beam, eccentric from the beam-column joint. In frames with partial-height masonry infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay.

6.2.2 *Stiffness of concrete frames with masonry infills*

6.2.2.1 *Linear static and dynamic procedures*—In frames having infills in some bays and no infill in other bays, the restraint of the infill shall be represented as described in 6.2.1. Bays without infills shall be modeled as frames as specified in appropriate portions of Chapters 4, 5, and 9. Where infills are discontinuous over the height, the effects of the discontinuity on overall building performance shall be evaluated. Effective stiffnesses shall be in accordance with 3.1.2.

6.2.2.2 *Nonlinear static procedure*—Nonlinear load-deformation relations for use in analysis by the NSP shall follow the requirements of 3.1.2.2. Modeling beams and columns using nonlinear truss elements shall be permitted in infilled portions of the frame. Beams and columns in non-infilled portions of the frame shall be modeled using the relevant specifications of Chapters 4, 5, and 9. The model shall be capable of representing inelastic response along the component lengths.

Monotonic load-deformation relations shall be according to the generalized relation shown in Fig. 1, except different relations shall be permitted where verified by tests. Numerical quantities in Fig. 1 shall be derived from tests or by analytical procedures, as specified in Chapter 7 of ASCE 41-17, and shall take into account the interaction between frame and infill components. Alternatively, the following procedure shall be permitted for monolithic reinforced concrete frames:

a) For beams and columns in bays without infills, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, the plastic hinge rotation capacities shall be as defined by Tables 7 and 8.

b) For masonry infills, the generalized deformations and control points shall be as defined in Chapter 11 of ASCE 41-17.

c) For beams and columns in bays with infills, where the generalized deformation is taken as elongation or compression displacement of the beams or columns, the tension and compression strain capacities shall be as specified in Table 17.

6.2.2.3 *Nonlinear dynamic procedure*—Nonlinear load-deformation relations for use in analysis by NDP shall model the complete hysteretic behavior of each component

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STANDARD COMMENTARY

Table 17—Modeling parameters and numerical acceptance criteria for nonlinear procedures—reinforced concrete infilled frames

Conditions	Modeling parameters*			Acceptance criteria		
	Total strain		Residual strength ratio	Total strain		
				IO	Performance level	
	d	e	c		LS	CP
i: Columns modeled as compression chords†						
Columns confined along entire length‡	0.02	0.04	0.4	0.003	0.03	0.04
All other cases	0.003	0.01	0.2	0.002	0.01	0.01
ii: Columns modeled as tension chords†						
Columns with well-confined splices or no splices	0.05	0.05	0.0	0.01	0.04	0.05
All other cases	§	0.03	0.2	—	0.02	0.03

*Interpolation shall not be permitted.

†If load reversals result in both Conditions i and ii applying to a single column, both conditions shall be checked.

‡A column shall be permitted to be considered confined along its entire length where the quantity of hoops along the entire story height including the joint is equal to three-fourths of that required by ACI 318M for boundary components of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_b$.

§Potential for splice failure shall be evaluated directly to determine the modeling and acceptance criteria. For these cases, refer to the generalized procedure of 6.3.2.

using properties verified by tests. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

6.2.3 *Strength of concrete frames with masonry infills*—Strengths of reinforced concrete components shall be calculated according to the general requirements of 3.2, as modified by other provisions of this standard. Strengths of masonry infills shall be calculated according to the requirements of Chapter 11 of ASCE 41-17. Strength calculations shall consider the following:

a) Limitations imposed by beams, columns, and joints in noninfilled portions of frames

b) Tensile and compressive capacity of columns acting as boundary components of infilled frames

c) Local forces applied from the infill to the frame

d) Strength of the infill

e) Connections with adjacent components

6.2.4 *Acceptance criteria for concrete frames with masonry infills*

6.2.4.1 *Linear static and dynamic procedures*—All component actions shall be classified as either deformation-controlled or force-controlled, as defined in 7.5.1 of ASCE 41-17. In primary components, deformation-controlled actions shall be restricted to flexure and axial actions in beams, slabs, and columns, and lateral deformations in masonry infill panels. In secondary components, deformation-controlled actions shall be restricted to those actions identified for the isolated frame in Chapters 4, 5, and 9, as appropriate, and for the masonry infill in 11.4 of ASCE 41-17.

Design actions shall be determined as prescribed in Chapter 7 of ASCE 41-17. Where calculated DCR values

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STANDARD

COMMENTARY

Table 18—Numerical acceptance criteria for linear procedures—reinforced concrete infilled frames

Conditions	*m*-factors*				
	Performance level				
	IO	Component type			
		Primary		Secondary	
		LS	CP	LS	CP
i: Columns modeled as compression chords†					
Columns confined along entire length‡	1	3	4	4	5
All other cases	1	1	1	1	1
ii: Columns modeled as tension chords†					
Columns with well-confined splices or no splices	3	4	5	5	6
All other cases	1	2	2	3	4

*Interpolation shall not be permitted.

†If load reversals result in both Conditions i and ii applying to a single column, both conditions shall be checked.

‡A column is permitted to be considered confined along its entire length where the quantity of hoops along the entire story height, including the joint, is equal to three-fourths of that required by ACI 318M for boundary components of concrete shear walls. The maximum longitudinal spacing of sets of hoops shall not exceed either $h/3$ or $8d_b$.

exceed unity, the following design actions shall be determined using limit analysis principles as prescribed in Chapter 7 of ASCE 41-17: 1) moments, shears, torsions, and development and splice actions corresponding to development of component strength in beams, columns, or masonry infills; and 2) column axial load corresponding to development of the flexural capacity of the infilled frame acting as a cantilever wall.

Design actions shall be compared with strengths in accordance with 7.5.2.2 of ASCE 41-17.

Values of *m*-factors shall be as specified in 11.4.2.4 of ASCE 41-17 for masonry infills; applicable portions of Chapters 4, 5, and 9 for concrete frames; and Table 18 for columns modeled as tension and compression chords. Those components that have design actions less than strengths shall be assumed to satisfy the performance criteria for those components.

6.2.4.2 *Nonlinear static and dynamic procedures*—In the design model, inelastic response shall be restricted to those components and actions that are permitted for isolated frames as specified in Chapters 4, 5, and 9, and for masonry infills as specified in 11.4 of ASCE 41-17.

Calculated component actions shall satisfy the requirements of 7.5.3.2 of ASCE 41-17 and shall not exceed the numerical values listed in Table 17; the relevant tables for isolated frames given in Chapters 4, 5, and 9; and the relevant tables for masonry infills given in Chapter 11 of ASCE 41-17. Component actions not listed in Tables 7, 8, and 10 shall be treated as force-controlled. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

6.2.5 *Retrofit measures for concrete frames with masonry infills*—Seismic retrofit measures for concrete frames with masonry infills shall meet the requirements of 3.7 and other provisions herein.

C6.2.5 *Retrofit measures for concrete frames with masonry infills*—The retrofit measures described in relevant commentary of Chapters 4, 5, and 9 for isolated frames, and retrofit measures described in relevant commentary of

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11.4 of ASCE 41-17 for masonry infills, can also be effective in retrofitting concrete frames with masonry infills. The licensed design professional is referred to FEMA 308 for further information in this regard. In addition, the following retrofit measures can be effective in rehabilitating concrete frames with infills:

a) Post-tensioning existing beams, columns, or joints using external post-tensioned reinforcement. Vertical post-tensioning can be effective in increasing tensile capacity of columns acting as boundary zones. Anchorages should be located away from regions where inelastic action is anticipated and should be designed considering possible force variations caused by seismic forces.

b) Modification of the element by selective material removal from the existing element. Either the infill should be completely removed from the frame or gaps should be provided between the frame and the infill. In the latter case, the gap requirements of Chapter 11 of ASCE 41-17 should be satisfied and adequate measures must be taken to guarantee the out-of-plane stability of the infill.

c) Changing the building system to reduce the demands on the existing element. Examples include the addition of supplementary seismic-force-resisting elements such as walls, steel braces, or buttresses; seismic isolation; and mass reduction.

6.3—Concrete frames with concrete infills

6.3.1 *General*—The analytical model for a concrete frame with concrete infills shall represent the strength, stiffness, and deformation capacity of beams, slabs, columns, beam-column joints, concrete infills, and all connections and components of the elements. Potential failure in flexure, shear, anchorage, reinforcement development, or crushing at any section shall be considered. Interaction with nonstructural components shall be included.

The analytical model shall be established considering the relative stiffness and strength of the frame and the infill, as well as the level of deformations and associated damage. For low deformation levels, and for cases where the frame is relatively flexible, the infilled frame shall be permitted to be modeled as a shear wall, with openings modeled where they occur. In other cases, the frame-infill system shall be permitted to be modeled using a braced-frame analogy such as that described for concrete frames with masonry infills in 6.2.

Frame components shall be evaluated for forces imparted to them through interaction of the frame with the infill as specified in Chapter 11 of ASCE 41-17. In frames with full-height infills, the evaluation shall include the effect of strut compression forces applied to the column and beam eccentric from the beam-column joint. In frames with partial-height infills, the evaluation shall include the reduced effective length of the columns above the infilled portion of the bay.

In frames with infills in only some bays, the restraint of the infill shall be represented as described in this section. Bays without infills shall be modeled as frames as specified in appropriate portions of Chapters 4, 5, and 9. Where infills create a discontinuous wall over the height, the effects of

C6.3—Concrete frames with concrete infills

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STANDARD

COMMENTARY

the discontinuity on overall building performance shall be evaluated.

6.3.2 *Stiffness of concrete frames with concrete infills*

6.3.2.1 *Linear static and dynamic procedures*—Effective stiffnesses shall be calculated according to the principles of 3.1.2.1 and the procedure of 6.2.2.1.

6.3.2.2 *Nonlinear static procedure*—Nonlinear load-deformation relations for use in analysis by NSP shall follow the requirements of 3.1.2.2.

Monotonic load-deformation relations shall be according to the generalized relation shown in Fig. 1, except that different relations shall be permitted where verified by tests. Numerical quantities in Fig. 1 shall be derived from tests or by analysis procedures specified in 7.6 of ASCE 41-17 and shall take into account the interactions between frame and infill components. Alternatively, the procedure of 4.2.2.2 shall be permitted for the development of nonlinear modeling parameters for concrete frames with concrete infills.

6.3.2.3 *Nonlinear dynamic procedure*—Nonlinear load-deformation relations for use in analysis by NDP shall model the complete hysteretic behavior of each component using properties verified by tests. Unloading and reloading properties shall represent stiffness and strength degradation characteristics.

6.3.3 *Strength of concrete frames with concrete infills*—Strengths of reinforced concrete components shall be calculated according to the general requirements of 4.2, as modified by other specifications of this chapter. Strength calculations shall consider the following:

a) Limitations imposed by beams, columns, and joints in unfilled portions of frames

b) Tensile and compressive capacity of columns acting as boundary components of infilled frames

c) Local forces applied from the infill to the frame

d) Strength of the infill

e) Connections with adjacent components

Strengths of existing concrete infills shall be determined considering shear strength of the infill panel. For this calculation, procedures specified in 7.2.3 shall be used for calculation of the shear strength of a wall segment.

Where the frame and concrete infill are assumed to act as a monolithic wall, flexural strength shall be based on continuity of vertical reinforcement in both the columns acting as boundary components and the infill wall, including anchorage of the infill reinforcement in the boundary frame.

6.3.4 *Acceptance criteria for concrete frames with concrete infills*—The acceptance criteria for concrete frames with concrete infills shall comply with relevant acceptance criteria of 6.2.4, Chapter 7, and Chapter 8.

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STANDARD

6.3.5 *Retrofit measures for concrete frames with concrete infills*—Seismic retrofit measures for concrete frames with concrete infills shall meet the requirements of 3.7 and other provisions of this standard and ASCE 41.

COMMENTARY

C6.3.5 *Retrofit measures for concrete frames with concrete infills*—Retrofit measures described in C6.2.5 for concrete frames with masonry infills can also be effective in rehabilitating concrete frames with concrete infills. In addition, application of shotcrete to the face of an existing wall to increase the thickness and shear strength can be effective. For this purpose, the face of the existing wall should be roughened, a mat of steel reinforcement should be doweled into the existing structure, and shotcrete should be applied to the desired thickness. The licensed design professional is referred to FEMA 308 for further information regarding retrofit of concrete frames with concrete infill.

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STANDARD

COMMENTARY

CHAPTER 7—CONCRETE STRUCTURAL WALLS

7.1—Types of concrete structural walls and associated components

The provisions of Chapter 7 shall apply to all reinforced concrete structural walls in all types of structural systems that incorporate reinforced concrete structural walls. These types include isolated structural walls, structural walls used in wall-frame systems, coupled structural walls, and discontinuous structural walls. Structural walls shall be permitted to be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated structural walls shall be defined as walls that have a regular pattern of openings in both horizontal and vertical directions that creates a series of wall pier (vertical wall segment) and deep beam components (horizontal wall segment).

Coupling beams shall comply with provisions of 7.2 and shall be exempted from the provisions for beams covered in Chapter 4.

C7.1—Types of concrete structural walls and associated components

Concrete structural walls are planar vertical elements or combinations of interconnected planar elements that serve as lateral-load-resisting elements in concrete structures. Structural walls (or wall segments) shall be considered slender if their aspect ratio (h_w/ℓ_w [height/length]) is greater than 3.0 and shall be considered short or squat if their aspect ratio is less than 1.5. Slender walls are normally controlled by flexural behavior; short walls are normally controlled by shear behavior. The response of walls with intermediate aspect ratios is influenced by both flexure and shear.

Identification of component types in concrete structural wall elements depends, to some degree, on the relative strengths of the wall segments based on expected or measured material properties. Vertical segments are often termed “wall piers”, whereas horizontal segments can be called “coupling beams” or “spandrels”. The licensed design professional is referred to FEMA 306 for additional information regarding the behavior of concrete wall components. Selected information from FEMA 306 has been reproduced in Table C3 and Fig. C3 to clarify wall component identification.

7.1.1 *Monolithic reinforced concrete structural walls and wall segments*—Monolithic reinforced concrete structural walls shall consist of vertical cast-in-place elements, either uncoupled or coupled, in open or closed shapes. These walls shall have relatively continuous cross sections and reinforcement and shall provide both vertical and lateral force resistance, in contrast with infilled walls defined in 6.1.3.

Structural walls or wall segments with axial loads greater than $0.35P_o$ shall not be considered effective in resisting seismic forces. For the purpose of determining effectiveness of structural walls or wall segments, the use of axial loads based on a limit state analysis shall be permitted.

C7.1.1 *Monolithic reinforced concrete structural walls and wall segments*—The wall reinforcement is normally continuous in both the horizontal and vertical directions, and bars are typically lap-spliced for tension continuity. The reinforcement mesh can also contain horizontal ties around vertical bars that are concentrated either near the vertical edges of a wall with constant thickness or in boundary members formed at the wall edges. The amount and spacing of these ties is important for determining how well the concrete at the wall edge is confined and, thus, for determining the lateral deformation capacity of the wall.

In general, slender reinforced concrete structural walls are governed by flexure and tend to form a plastic flexural hinge near the base of the wall under severe lateral loading. The ductility of the wall is a function of the percentage of longitudinal reinforcement concentrated near the bound-

Table C3—Reinforced concrete shear wall component types

Component type per FEMA 306		Description	ASCE 41 designation
RC1	Isolated wall or stronger wall pier	Stronger than beam or spandrel components that can frame into it so that nonlinear behavior (and damage) is generally concentrated at the base, with a flexural plastic hinge or shear failure. Includes isolated (cantilever) walls. If the component has a major setback or cutoff of reinforcement above the base, this section should be also checked for nonlinear behavior.	Monolithic reinforced concrete wall or vertical wall segment
RC2	Weaker wall pier	Weaker than the spandrels to which it connects; characterized by flexural hinging top and bottom or shear failure.	
RC3	Weaker spandrel or coupling beam	Weaker than the wall piers to which it connects; characterized by hinging at each end, shear failure, or sliding shear failure.	Horizontal wall segment or coupling beam
RC4	Stronger spandrel	Should not suffer damage because it is stronger than attached wall piers. If this component is damaged, it should probably be reclassified as RC3.	
RC5	Pier-spandrel panel zone	Typically not a critical area in RC walls.	Wall segment

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COMMENTARY

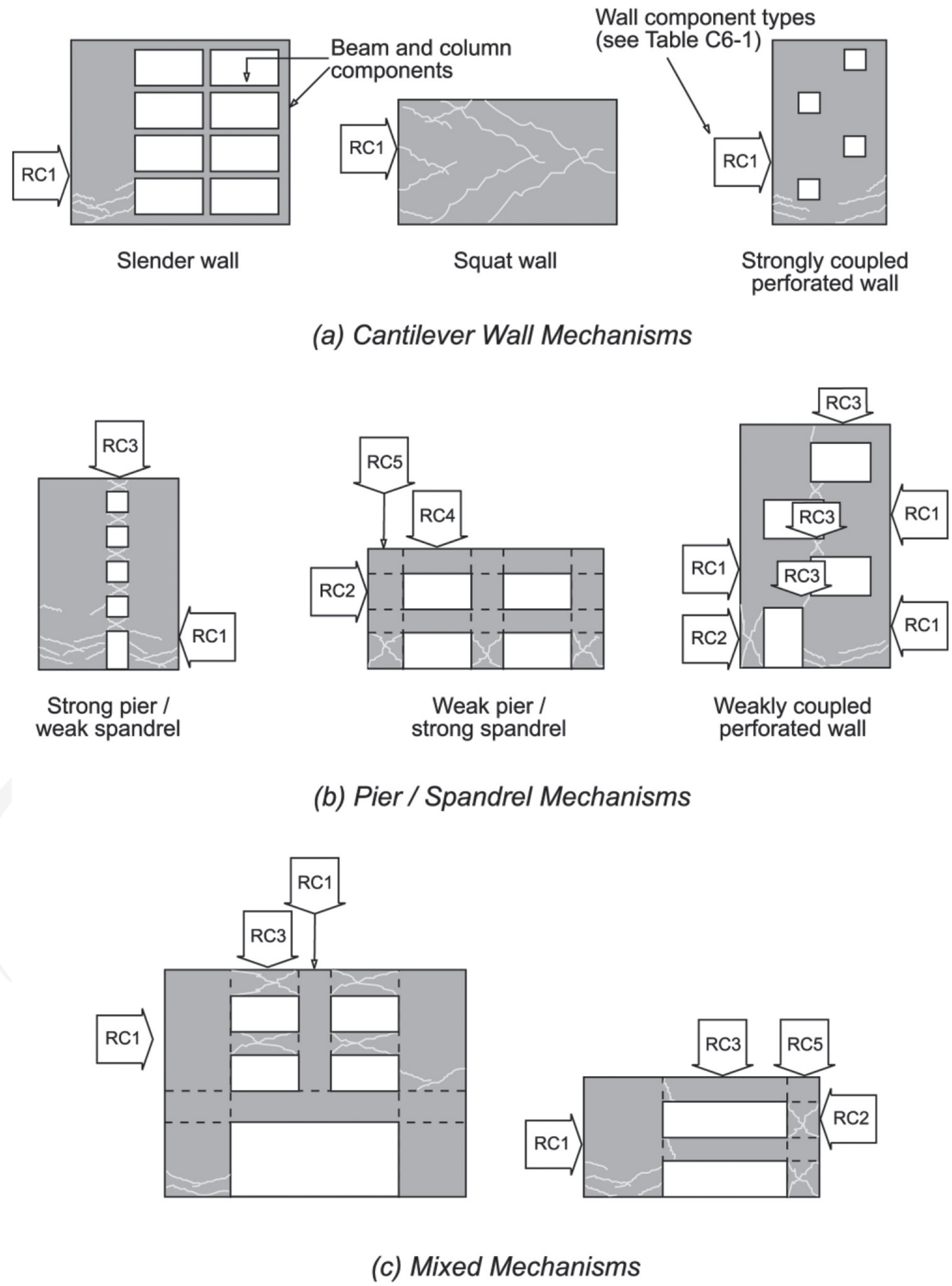

Fig. C3—Identification of component types in concrete shear wall elements (FEMA 36).

aries of the wall, level of axial load, amount of lateral shear required to cause flexural yielding, thickness, reinforcement used in the web portion of the shear wall, and transverse reinforcement in the boundary elements, including the ratio of the transverse reinforcement spacing to the diameter of the longitudinal reinforcing bars. In general, higher axial load stresses and higher shear stresses reduce the flexural ductility and energy-absorbing capability of the wall. Short or squat structural walls are normally governed by shear. These walls normally have a limited ability to deform beyond the elastic range and continue to resist seismic forces. Thus, these walls are typically analyzed either as displacement-

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STANDARD

COMMENTARY

controlled components with low ductility capacities or as force-controlled components.

7.1.2 *Reinforced concrete columns supporting discontinuous structural walls*—Reinforced concrete columns supporting discontinuous structural walls shall be analyzed in accordance with the requirements of 4.2.

C7.1.2 *Reinforced concrete columns supporting discontinuous structural walls*—In structural wall buildings, it is not uncommon to find that some walls are terminated either to create commercial space in the first story or to create parking spaces in the basement. In such cases, the walls are commonly supported by columns. Such designs are not recommended in seismic zones because very large demands can be placed on these columns during earthquake loading. In older buildings, such columns often have standard longitudinal and transverse reinforcement; the behavior of such columns during past earthquakes indicates that tightly spaced closed ties with well-anchored 135-degree hooks are required for the building to survive severe seismic forces.

7.1.3 *Reinforced concrete coupling beams*—Reinforced concrete coupling beams used to link two shear walls together shall be evaluated and rehabilitated to comply with the requirements of 7.2.

C7.1.3 *Reinforced concrete coupling beams*—Coupled walls are generally much stiffer and stronger than they would be if they acted independently. Coupling beams typically have a small span-depth ratio, and their inelastic behavior is normally affected by the high shear forces acting in these components. Coupling beams in most older reinforced concrete buildings commonly have conventional reinforcement that consists of longitudinal flexural steel and transverse steel for shear. In some more modern buildings, or in buildings where coupled structural walls are used for seismic retrofit, the coupling beams can use diagonal reinforcement as the primary reinforcement for both flexure and shear. The inelastic behavior of coupling beams that use diagonal reinforcement has been shown experimentally to be much better with respect to retention of strength, stiffness, and energy dissipation capacity than the observed behavior of coupling beams with nonprestressed reinforcement.

7.2—Reinforced concrete structural walls, wall segments, and coupling beams

7.2.1 *General*—The analytical model for a structural wall element shall represent the stiffness, strength, and deformation capacity of the wall. Potential failure in flexure, shear, and reinforcement development at any point in the wall shall be considered. Interaction with other structural and nonstructural components shall be included.

Slender structural walls and wall segments shall be permitted to be modeled as equivalent beam-column elements that include both flexural and shear deformations. The flexural strength of beam-column elements shall include the interaction of axial load and bending, and shall be calculated based on expected material properties. The rigid connection zone at beam connections to this equivalent beam-column element shall represent the distance from the wall centroid to the edge of the wall. Unsymmetrical wall sections shall be modeled with the different bending capacities for the two loading directions.

A beam element that incorporates both bending and shear deformations shall be used to model coupling beams. The

C7.2—Reinforced concrete structural walls, wall segments, and coupling beams

C7.2.1 *General*—For rectangular structural walls, wall segments with $h_w/\ell_w \leq 2.5$ and flanged wall sections with $h_w/\ell_w \leq 3.5$, either a modified beam-column analogy or a multiple-node, multiple-spring approach should be used. Because structural walls usually respond in single curvature over a story height, one multiple-spring element per story can be used for modeling walls. Wall segments should be modeled with either the beam-column element or with a multiple-spring model with two elements over the length of the wall segment.

Coupling beams that have diagonal reinforcement satisfying ACI 318M requirements commonly have a stable hysteretic response under large load reversals. Therefore, these members could adequately be modeled with beam elements used for typical frame analyses.

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