ACI ITG-5.1M-07

Acceptance Criteria for Special Unbonded Post-Tensioned Precast Structural Walls Based on Validation Testing and Commentary

An ACI Standard

Reported by ACI Innovation Task Group 5

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This document applies to structures in regions of high seismic risk or to structures assigned to high seismic performance or design categories. It defines the minimum experimental evidence that can be deemed to satisfy the use of unbonded post-tensioned precast structural walls (shear walls) for bearing wall and building frame special reinforced concrete shear wall systems, as defined in ASCE/SEI 7-05, when those walls do not fully satisfy the intent of the prescriptive requirements of Chapter 21 of ACI 318M-05. This document includes mandatory Acceptance Criteria and nonmandatory Commentary, and has been written in such a form that its requirements can be coordinated directly with the requirements for special precast structural walls in 21.8 of ACI 318M-05. Among the subjects covered are requirements for the procedures that shall be used to design unbonded post-tensioned precast test modules and their configurations, as well as requirements for testing, reporting, and assessing satisfactory performance of the test modules.

The references of the Commentary provide documentary evidence, additional to the references of Chapter 21 of ACI 318RM-05, that support the acceptance criteria. Consistent with the approach of ACI 318M, no comparison is made, either in the body of the Acceptance Criteria or Commentary, of research results for precast test modules satisfying ACI 318M with those for modules that, although not satisfying ACI 318M, do satisfy the Acceptance Criteria. Such comparisons, both experimental and analytical, are available in the Commentary references.

In this document, consistent with the format of ACI 318M-05, the word "Section" is not included before a reference to a section of ACI 318M-05. To more clearly designate a section in this document, however, the word "Section" is used before any reference to a section of this document.

The section numbering for the Commentary is the same as that for the Standard, with numbers preceded by an "R" and the text in italics to distinguish them from the corresponding section numbers of the Standard.

Keywords: acceptance criteria; coupling element; drift; drift angle; energy dissipation; lateral resistance; post-tensioning; precast concrete; prestressed concrete; seismic design; shear wall; structural wall; test module; toughness.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

For seismic design, 21.2.1.5 of ACI 318M specifies that "a reinforced concrete structural system not satisfying the requirements of this chapter (Chapter 21) shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system has strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter." This document defines the minimum experimental evidence required to validate the use of special unbonded post-tensioned precast structural wall systems in regions of high seismic risk or for structures assigned to high seismic performance or design categories when those systems do not satisfy fully the prescriptive requirements of Chapter 21 of ACI 318M. The provisions of this document are intended to supplement the provisions of Chapter 21 of ACI 318M and not to supplant them.

Consistent with the 21.2.1.5 requirement of ACI 318M, this document specifies that, before the validation testing mandated by the document is undertaken, a design procedure shall have been developed for prototype unbonded post-tensioned precast structural walls having the generic form for which acceptance is sought. Further, the same design procedure shall be used to proportion the test modules. The document also requires that the prototype buildings that contain the unbonded post-tensioned precast structural walls have proportions that are essentially regular in the vertical direction, having no significant physical discontinuities in plan, in vertical configuration, or in their lateral-force-resisting systems.

This document is intended for walls that might, for example, involve the use of precast elements, precast prestressed elements, post-tensioned reinforcement, or combinations of those elements and reinforcement. Comprehensive prescriptive requirements for unbonded post-tensioned precast structural walls constructed with such elements are not included in ACI 318M.

1.2—Scope and general requirements

R1.2—Scope and general requirements

While only ACI Committee 318 can specify the requirements necessary for unbonded post-tensioned precast walls to meet the provisions of 21.2.1.5 of ACI 318M, 1.4 of ACI 318M permits the building official to accept precast wall systems, other than those explicitly covered by Chapter 21 of ACI 318M, provided specific tests, load factors, deflection limits, construction procedures, and other pertinent requirements have been established for acceptance of such systems consistent with the intent of the Code. This document provides a framework that establishes the specific tests, load factors, deflection limits, and other pertinent requirements appropriate for acceptance for regions of high seismic risk or for structures assigned to high seismic performance or design categories of unbonded post-tensioned precast wall systems, including unbonded post-tensioned precast coupled wall systems, not satisfying all the prescriptive requirements of Chapter 21 of ACI 318M.

This document assumes that the unbonded post-tensioned precast wall system to be tested has details that differ from those prescribed by 21.7 of ACI 318M for conventional monolithic reinforced concrete construction. Such walls may, for example, involve the use of precast elements, precast prestressed elements, post-tensioned reinforcement, or combinations of those elements and reinforcement. Life safety and toughness are theoretically enhanced by mild steel reinforcement grouted across the wall to foundation interface. The presence of mild steel reinforcement, however, makes erection more difficult, and may inhibit the self-centering action provided by unbonded post-tensioning crossing the same interface. For an uncoupled wall, mild steel reinforcement, or some other form of energy dissipating base connection, is necessary to meet the relative energy dissipation requirements of this document. For coupled walls, however, energy-dissipating coupling elements can be used along the vertical boundaries between walls so that only unbonded post-tensioning tendons need to cross the wall to foundation interface. Life safety for coupled walls is then more critically dependent on the unbonded post-tensioning not fracturing under the seismic event. In that case, careful attention should be paid to corrosion protection of the tendon and to stress increases in the tendon during the seismic event.

For monolithic reinforced concrete walls, a fundamental design concept underlying the Chapter 21 provisions of ACI 318M is that walls with $\mathbf{h_w/\ell_w}$ *exceeding 1.0 should be proportioned so that their inelastic response is dominated by flexural action on a critical section located near the base of the wall. That same basic fundamental concept is retained in this document. The limiting* $\mathbf{h_w/\ell_w}$ *value, however, is reduced to 0.5. The basis for that lower limit is discussed in R1.2.2.*

Tests on modules, as envisioned in this document, cannot be extrapolated with confidence to the performance of panelized walls of proportions differing from those tested for the development of Chapter 21 of ACI 318M if the shear-slip displacement pattern or excessive joint opening pattern of Fig. R2.3 is significant in the response developed in the test on the module.

Two other fundamental requirements of Chapter 21 of ACI 318M are for closely spaced ties around heavily strained boundary element reinforcement and the provision of minimum amounts of uniformly distributed horizontal and vertical reinforcement in the web of the wall. Ties around boundary element reinforcement to inhibit its buckling in compression are required where the strain in the extreme compression fiber exceeds 0.003 and spalling of the cover concrete may occur. Those ties then provide confinement that maintains the integrity of the boundary element and permits the confined concrete to develop increasing compressive forces with increasing lateral displacements in spite of the loss of the concrete cover. Minimum amounts of uniformly distributed horizontal and vertical reinforcement over the height and length of the wall are required to restrain the opening of inclined cracks and allow the development of *the minimum acceptable drift angle capacities specified in Section 5.4. Deviations from those tie and distributed reinforcement requirements are possible only if a theory is developed that can substantiate reasons for such deviations and that theory is tested as part of the validation testing.*

1.2.1 This document defines minimum acceptance criteria for unbonded post-tensioned precast structural walls, including coupled unbonded post-tensioned precast structural walls, designed for regions of high seismic risk or for structures assigned to high seismic performance or design categories, where acceptance is based on experimental evidence and analysis.

R1.2.1 *This document is not intended for use directly with existing construction or for use with walls that are designed to conform to all the requirements of 21.7 of ACI 318M. The criteria of this document are at least as stringent as those for walls designed to the minimum requirements of 21.7 of ACI 318M. Some walls designed to 21.7, and having low height-length ratios, may not meet the minimum acceptable drift angle capacity of Eq. (5-1) because their behavior may be governed almost entirely by shear deformations (Hidalgo et al. 2002). The height-length ratio of 0.5 is the least value for which Eq. (5-1) is applicable.*

1.2.2 This document is applicable to unbonded post-tensioned precast structural walls, coupled or uncoupled, with height-to-length ratios, h_w/ℓ_w, equal to or greater than 0.5. This document is applicable to either prequalifying unbonded post-tensioned precast structural walls for a specific structure or prequalifying an unbonded post-tensioned precast wall type for construction in general. This document applies to walls that are effectively continuous from the base of the structure to top of wall and are designed to have a single critical section for flexure and axial loads.

R1.2.2 *The use of this document is limited to walls with* $\mathbf{h_w/\ell_w}$ *values of 0.5 or greater primarily for two reasons. First, for rectangular walls, elastic cracked section flexural deformations start to exceed elastic cracked section shear deformations once the* $\mathbf{h_w/\ell_w}$ *value exceeds 0.5. Second, experimental evidence shows that postpeak load drift values for walls with* $\mathbf{h_w/\ell_w}$ *values less than 0.5 are unreliable unless the walls contain considerably more horizontal and vertical reinforcement than the 0.25% minimum required by 21.7.2.1 in ACI 318M (Hidalgo et al. 2002). Further, for precast walls with grouted joints between panels, the likelihood of sliding increases as the* $\mathbf{h_w/\ell_w}$ *value decreases and the presence of post-tensioning becomes increasingly essential in ensuring nonsliding behavior. For walls with* $\mathbf{h_w/\ell_w}$ *values of 0.5 or greater, postpeak load drifts are about 0.5% regardless of the* $\mathbf{h_w/\ell_w}$ *value of the wall (Hidalgo et al. 2002). From Eq. (5-1), it follows that for walls with* $\mathbf{h_w/\ell_w}$ *values of 0.5, only about 0.4% drift angle is contributed by the prepeak load response. If the postpeak load drift is unreliable, the drift capacity becomes unreliable for* $\mathbf{h_w/\ell_w}$ *values less than 0.5. Limiting the use of this document to walls with* $\mathbf{h_w/\ell_w}$ *values of 0.5 and greater does not imply that the behavior of walls with* $\mathbf{h_w/\ell_w}$ *values less than 0.5 is unreliable. Rather, the behavior of such walls can be very reliable if shear stresses are low and the wall is not required to deform to displacements greater than the displacement associated with the peak load capacity of the wall.*

The wall heights to which this document can be applied are not limited because a practical limit will be effectively imposed by the combination of testing facility restrictions on height and the minimum specimen scale limits imposed in this document. Further, for heights greater than about 30 m and the customary $\mathbf{h_w/\ell_w}$ *ratios for precast panels, the design displacements result in stress level changes in the prestressing steel that are difficult to accommodate while still maintaining the desirable self-centering characteristic of an unbonded post-tensioned wall. For coupled walls, the situation becomes even more severe because the prestressing steel stress changes are even greater for the trailing wall than would be the case for an uncoupled wall of the same height (Thomas and Sritharan 2004).*

Consistent with the concept of 21.7.6.2 of ACI 318M, procedures are based on the assumption that inelastic response of the wall is dominated by flexure at a single critical jointed section that is a potential yielding section. The wall should be proportioned so that the critical section occurs where intended. For walls with openings, the influence of an opening or openings on flexural and shear strengths should be considered, and a load path around the opening or openings should be verified. The presence of the opening or openings should be verified to not affect the location of the critical jointed section that is the potential yielding section.

1.2.3 The walls of the prototype structure shall be permitted to be several panels long and several panels high, to be constructed from subpanels, and to be either coupled or uncoupled. Coupling elements shall be permitted to be devices or beams connecting adjacent vertical boundaries of the unbonded post-tensioned precast walls.

R1.2.3 *For uncoupled walls, relative energy dissipation ratios increase as the drift angle increases (Kurama 2002). Tests on slender monolithic walls have shown relative energy dissipation ratios, derived from rotations at the base of the wall, of approximately 40 to 45% at large drifts (Ali and Wight 1990). The same result has been reported even where there has been a significant opening in the web of the wall on the compression side (Taylor et al. 1998). Kurama (2002) computed relative energy dissipation ratios at a 0.020 radians drift angle for uncoupled walls with height-length ratios of 4. Ratios were 30, 18, 12, and 6% for monolithic reinforced concrete, hybrid reinforced/post-tensioned prestressed concrete with equal flexural strengths provided by the prestressed and deformed bar reinforcement, hybrid reinforced/post-tensioned prestressed concrete with 25% of the flexural strength provided by deformed bar reinforcement and 75% by the prestressed reinforcement, and post-tensioned prestressed concrete special structural walls, respectively. Thus, for slender precast uncoupled walls of emulative or nonemulative design, it should be anticipated that at least 27% of the flexural capacity at the base of the wall should be provided by deformed bar reinforcement if* *the requirement of a relative energy dissipation ratio of 1/8 is to be achieved. If more than approximately 40% of the flexural capacity at the base of the wall is provided by deformed bar reinforcement (Kurama 2002), however, then the self-centering capability of the wall following a major event is lost, and that is one of the benefits gained with the use of post-tensioning. For squat walls with height-length ratios between 0.35 and 0.69, the relative energy dissipation has been reported (Hildalgo et al. 2002) as remaining constant at 23% for drifts between that for first diagonal cracking and that for a postpeak capacity of 80% of the peak capacity. Thus, regardless of whether the behavior of a wall is controlled by shear or flexural deformations, a minimum relative energy dissipation ratio of 1/8 is a realistic requirement.*

For an unbonded post-tensioned wall, it is difficult to separate the strength provided by the post-tensioning from the strength provided by the deformed bar reinforcement and the strength provided by axial load because both the post-tensioning and axial load move the neutral axis toward the center of the wall and increase the moment strength. If self-centering is a goal, it is recommended that the engineer developing the system calculate independently the deformed bar reinforcement requirements because the values of the previous paragraph provide guidance only.

For coupled wall systems, theoretical studies (Stanton and Nakaki 2002) and tests (Priestley et al. 1999) have demonstrated that the 1/8 relative energy dissipation ratio can be achieved by using central post-tensioning only in the walls and appropriate energy-dissipating coupling devices connecting adjacent vertical wall boundaries.

1.2.4 Unbonded post-tensioned precast structural walls shall be deemed to have a strength and toughness that is adequate to comply with 21.2.1.5 of ACI 318M, and the corresponding unbonded post-tensioned precast structural walls of the prototype structure shall be deemed acceptable when all of the conditions in Sections 1.2.4.1 through 1.2.4.5 are satisfied.

1.2.4.1 The prototype structure satisfies all applicable requirements of this document and of ACI 318M except 21.7.

R1.2.4.1 *The precast walls should meet the strength, minimum reinforcement, and detailing requirements of Chapters 11 and 14 of ACI 318M as well as the structural integrity requirements of Chapter 16. The structural integrity provisions of Chapter 16 require a minimum of two ties per panel with a nominal strength of not less than 44 kN per tie. Thus, at least two tendons per wall, or one tendon and one grouted in-place deformed steel tie bar, are required.*

By not requiring the walls to meet 21.7 of ACI 318M, the specific triggers of 21.7.6 of ACI 318M concerning the need for boundary elements in special structural walls are rescinded. This does not mean, however, that boundary elements or confinement reinforcement in wall boundaries will not be required. The need for such elements and reinforcement for jointed walls should be explored using displacement-based concepts consistent with the concepts of 21.7.6.2 for monolithic walls.

1.2.4.2 Tests on wall modules satisfy the conditions in Chapters 3 and 7.

1.2.4.3 The prototype structure is designed using the design procedure substantiated by the testing program.

1.2.4.4 The prototype structure is:

(1) Analyzed using stiffness properties consistent with those validated as prescribed in Section 5.10;

(2) Demonstrated through analysis to have design displacements for all walls equal to or less than two thirds of the displacement associated with the drift angles to which the appropriate modules have been tested in accordance with Section 5.4; and

(3) Shown to have factored engineering design values for all walls less than the nominal engineering design values used for the test modules in accordance with Section 4.2.

R1.2.4.4 *Both IBC 2003 and NFPA 5000 contain allowable story drift limits. In IBC 2003, allowable story drifts* $\Delta_{\mathbf{a}}$ *are specified in Table 1617.3, and likely values are discussed in the Commentary of Section 5.4 of this document. The limiting initial drift angle consistent with* $\Delta_{\mathbf{a}}$ *equals* $\Delta_{\mathbf{a}}/\phi\mathbf{C_d h_w}$, *where* ϕ *is the strength reduction factor appropriate to the condition, flexure or shear, that controls the design of the test module. For example, for* $\Delta_{\mathbf{a}}/\mathbf{h_w}$ *equal to 0.015, the required deflection amplification factor* $\mathbf{C_d}$ *of 5, and* ϕ *equal to 0.9, the limiting initial drift angle, corresponding to B in Fig. R7.1, is 0.0033. The use of a* ϕ *value is necessary because the allowable story drifts of the IBC are for the design seismic load effect* $\mathbf{E}$, *while the limiting initial drift angle is at the nominal strength* $\mathbf{E_n}$, *which should be greater than* $\mathbf{E}/\phi$. *The load-deformation relationship of a wall becomes significantly nonlinear before the applied load reaches* $\mathbf{E_{nt}}$. *While the load at which that nonlinearity becomes marked depends on the structural characteristics of the wall, the response of most walls remains linear up to approximately 75% of* $\mathbf{E_{nt}}$ *(Kurama et al. 1999).*

1.2.4.5 The structure as a whole, based on the results of the tests of Section 1.2.4.2 and analysis, is demonstrated to have the required global toughness.

R1.2.4.5 *The criteria of Chapter 7 are for the test module. In contrast, the criterion of Section 1.2.4.5 is for the structural system as a whole, and can be satisfied only by the details used for the design and analysis of the building as a whole. The criterion adopted herein is similar to that described in the last paragraph of R21.2.1 of ACI 318M. The intent is that test results and analyses demonstrate that the structure, after cycling three times through both positive and negative values of the minimum acceptable limiting drift angle capacity specified in Section 5.4 of this document, is still capable of supporting the gravity load specified as acting on the structure during the earthquake.*