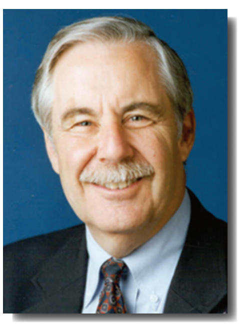
Acceptance Criteria for Special Precast Concrete Structural Walls Based on Validation Testing



Neil M. Hawkins, Ph.D.
Professor Emeritus
Department of Civil Engineering
University of Illinois at
Urbana-Champaign
Urbana, Illinois

S.K. Ghosh, Ph.D., FPCI
President
S.K. Ghosh Associates, Inc.
Palatine, Illinois

The 2003 Edition of the NEHRP Provisions permits, with validation testing, the use of structural (shear) walls of jointed construction in regions of high seismic risk. Acceptance criteria for such validation testing, accompanied by background and commentary, are presented in this article. Lastly, the future direction of non-emulative design of precast concrete walls is discussed.

In the July-August 2003 issue of the PCI JOURNAL,[1] the authors discussed PCI's strategy for codification of PRESSS structural systems with emphasis on the strategy for non-emulative design of special precast concrete shear walls. Such shear walls were used in one direction of the PRESSS five-story building tested at the University of California at San Diego.[2]

Code provisions for non-emulative design of special precast concrete shear walls have been lacking for many years. However, a recent PCI-initiated proposal is now included in the 2003 edition of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,[3] as noted in the last issue of the PCI JOURNAL.[4] The 2003 NEHRP Provisions and Commentary have been forwarded by the Building Seismic Safety Council (BSSC) to the Federal Emergency Management Agency (FEMA) and those documents are now in the public domain.

The 2003 NEHRP Provisions and Commentary are available for viewing and downloading on the BSSC website (www.bssconline.org). It is understood as of this writing, that for budgetary reasons, the Provisions and Commentary will not be made available in printed form. A CD version is expected to be available in the near-term future.

Some background information on the codification of non-emulative precast wall systems must first be provided.

BACKGROUND

For regions of high seismicity, Section 21.2.1.5 of ACI 318-02[5] permits the use of structural systems that do not meet the relevant prescriptive requirements of Chapter 21 if certain "experimental evidence and analysis" are provided. ACI Standard T1.1-01,[6] "Acceptance Criteria for Moment Frames Based on Structural Testing," defines the minimum evidence required when attempting to validate the use of strong column-weak beam moment frames in accordance with that section.

ACI T1.1-01 should be used when attempting to validate the wide variety of frames possible by using precast elements, precast prestressed elements, precast elements post-tensioned together, and combinations of such elements. Any combination of these elements can result in deformation, strength, energy absorption, and ductility characteristics different from those for monolithic reinforced concrete construction.

Before acceptance testing, 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 the design procedure be used to proportion the test modules. Provisional Standard ACI T1.2-03[7] defines the design procedure to be used for one specific type of moment frame (the so-called hybrid frame, in which features of post-tensioned and precast concrete construction are combined) that does not satisfy the requirements of Chapter 21 of ACI 318-02, but that can be validated for use in regions of high seismicity under ACI T1.1-01.

Special moment frames constructed using precast concrete and not emulating special moment frames of cast-in-place concrete are specifically permitted by ACI 318-02, provided they satisfy the requirements of ACI T1.1-01.

At a meeting of the PRESSS Advisory Group on May 30, 2001, it was decided to pursue the codification process for two additional structural systems selected out of the five that were used in the PRESSS five-story building test: the Pretensioned Precast Frame System and the Precast Shear Wall System.

The pretensioned precast frame system has been the subject of a recent article.[8] The precast shear wall system is discussed in the remainder of this article.

EVOLUTION

A proposed Provisional Standard and Commentary titled "Acceptance Criteria for Special Structural Walls Based on Validation Testing" was developed by Neil M. Hawkins and S.K. Ghosh in early 2003.[9] This document proposes the minimum experimental evidence that can be deemed adequate to attempt to validate, in regions of high seismic risk or in structures assigned to high seismic performance or design categories, the use of structural walls (shear walls), including coupled walls, for Bearing Wall and Building Frame Systems (Section 9 of ASCE 7-02), not fully satisfying the prescriptive requirements of Chapter 21 of ACI 318-02.

The document consists of both a Provisional Standard and a Commentary that is not part of the Provisional Standard. The document has been written in such a form that its various parts can be adopted directly into Sections 21.0, 21.1, and 21.2.1 of ACI 318-02 and the corresponding sections of ACI 318R-02. Among the subjects covered are requirements for procedures that must be used to design test modules; configurations for these modules; test methods; test reports; and determination of satisfactory performance.

Input on the above document was received at a PCI Review Group meeting at PCI Headquarters on January 31, 2003. A modified version, dated February 3, 2003, which accommodated the Review Group input, was presented at a meeting of BSSC Technical Subcommittee 4 on Concrete (TS4) in Portland, Oregon, on February 8, 2003. A letter ballot of the Technical Subcommittee was subsequently conducted. Further modifications were made in response to several valuable comments from Joe Maffei, a member of TS4.

The modified document was then balloted by the BSSC Provisions Update Committee (PUC) prior to their meeting in San Diego on June 15-17, 2003; the proposal drew a large number of negative votes. Considerable effort was spent to respond to every negative comment that was submitted. Further significant adjustments were made to the proposal at the PUC meeting.

With the modifications, the PCI-initiated proposal to permit non-emulative design of special precast concrete shear walls, using a modified version of "Acceptance Criteria for Special Structural Walls Based on Validation Testing," was approved by the PUC for inclusion in the 2003 edition of the NEHRP Provisions. Some relatively minor additional adjustments were made as a result of comments received from a letter ballot of the member organizations (including PCI) of BSSC.

The adjusted version now appears in the 2003 NEHRP Provisions. The extensive commentary that was included in Reference 9 is now part of the 2003 NEHRP Provisions Commentary. One result of all the input has been that the scope of the Acceptance Criteria and Commentary is now limited to special precast wall systems, to the exclusion of special cast-in-place walls.

The Acceptance Criteria and Commentary are intended for walls that might, for example, involve the use of precast concrete elements, precast/prestressed elements, post-tensioned reinforcement, or combinations of these elements and reinforcement. Comprehensive prescriptive requirements for special precast shear walls constructed with such elements are not included in ACI 318-02.

ACCEPTANCE CRITERIA

Section 21.8 of ACI 318-02, regarding special structural walls constructed using precast concrete, requires:

21.8.1 *– Special structural walls constructed using precast concrete shall satisfy all requirements of Section 21.7 for cast-in-place special structural walls in addition to Sections 21.13.2 and 21.13.3.*

The NEHRP Provisions permits non-emulative walls by introducing a Section 9.2.2.4 in Chapter 9 – Concrete Structure Design Requirements, as follows:

9.2.2.4 Special structural walls constructed using precast concrete. *Add a new Section 21.8.2 to ACI 318-02 as follows:*

"21.8.2 *– Wall systems not meeting the requirements of Section 21.8.1 shall be permitted if substantiating experimental evidence and analysis meets the requirements of Section 9.6."*

The new Section 9.6 reads as follows:

9.6 - ACCEPTANCE CRITERIA FOR SPECIAL PRECAST STRUCTURAL WALLS BASED ON VALIDATION TESTING

9.6.1 – Notation:

Symbols additional to those in Chapter 21 of ACI 318 are defined as:

E_{max} = *maximum lateral resistance of test module determined from test results (forces or moments)*

E_n = *nominal lateral resistance of test module calculated using specified geometric properties of test members, specified yield strength of reinforcement, specified compressive strength of concrete, a strain compatibility analysis or deformation compatibility analysis for flexural strength and a strength reduction factor ϕ of 1.0*

E_{nt} = *Calculated lateral resistance of test module using the actual geometric properties of test members, the actual strengths of reinforcement, concrete, and coupling devices, obtained by testing per Sections 9.6.7.7, 9.6.7.8, and 9.6.7.9; and a strength reduction factor ϕ of 1.0*

θ = *drift ratio*

β = *relative energy dissipation ratio*

9.6.2 Definitions:

Definitions additional to those in Chapter 21 of ACI 318 are defined.

9.6.2.1 – Coupling Elements *– Devices or beams connecting adjacent vertical boundaries of structural walls and used to provide stiffness and energy dissipation for the connected assembly greater than the sum of those provided by the connected walls acting as separate units.*

9.6.2.2 – Drift ratio – *Total lateral deformation of the test module divided by the height of the test module.*

9.6.2.3 – Global toughness *– The ability of the entire lateral force-resisting system of the prototype structure to maintain structural integrity and continue to carry the required gravity load at the maximum lateral displacements anticipated for the ground motions of the maximum considered earthquake.*

9.6.2.4 – Prototype structure – *The concrete wall structure for which acceptance is sought.*

9.6.2.5 – Relative energy dissipation ratio - *Ratio of actual to ideal energy dissipated by test module during reversed cyclic response between given drift ratio limits, expressed as the ratio of the area of the hysteresis loop for that cycle to the area of the circumscribing parallelograms defined by the initial stiffnesses during the first cycle and the peak resistances during the cycle for which the relative energy dissipation ratio is calculated (see Section 9.6.9.1.3).*

9.6.2.6 – Test module - *Laboratory specimen representing the critical walls of the prototype structure (see Section 9.6.5).*

9.6.3 – Scope and General Requirements:

9.6.3.1 *– These provisions define minimum acceptance criteria for new precast structural walls, including coupled 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 mathematical analysis.*

9.6.3.2 *– These provisions are applicable to precast structural walls, coupled or uncoupled, with height-to-length ratios, h_w/l_w, equal to or greater than 0.5. These provisions are applicable for either prequalifying precast structural walls for a specific structure or prequalifying a new precast wall type for construction in general.*

9.6.3.3 *– Precast structural walls shall be deemed to have a response that is at least equivalent to the response of monolithic structural walls designed in accordance with Sections 21.2 and 21.7 of ACI 318, and the corresponding structural walls of the prototype structure shall be deemed acceptable, when all of the conditions in Sections 9.6.3.3.1 through 9.6.3.3.5 are satisfied:*

9.6.3.3.1 *– The prototype structure satisfies all applicable requirements of these provisions and of ACI 318, except Section 21.7.*

9.6.3.3.2 *– Tests on wall modules satisfy the conditions in Sections 9.6.4 and 9.6.9.*

9.6.3.3.3 *– The prototype structure is designed using the design procedure substantiated by the testing program.*

9.6.3.3.4 *– The prototype structure is designed and analyzed using effective initial properties consistent with those determined in accordance with Section 9.6.7.11, and the prototype structure meets the drift limits of these provisions.*

9.6.3.3.5 *– The structure as a whole, based on the results of the tests of Section 9.6.3.3.2 and analysis, is demonstrated to have adequate global toughness (the ability to retain its structural integrity and support its specified gravity loads) through peak displacements equal to or exceeding the story-drift ratios specified in Sections 9.6.7.4, 9.6.7.5 or 9.6.7.6, as appropriate.*

9.6.4 – Design Procedure:

9.6.4.1 *– Prior to testing, a design procedure shall be developed for the prototype structure and its walls. This procedure shall account for effects of material nonlinearity, including cracking, deformations of members and connections, and reversed cyclic loading. The design procedure shall include the procedures specified in Sections 9.6.4.1.1 through 9.6.4.1.4, and shall be applicable to all precast structural walls, coupled and uncoupled, of the prototype structure.*

9.6.4.1.1 *– Procedures shall be specified for calculating the effective initial stiffness of the precast structural walls, and of coupled structural walls, that are applicable to all the walls of the prototype structure.*

9.6.4.1.2 *– Procedures shall be specified for calculating the lateral strength of the precast structural walls, and of coupled structural walls, applicable to all precast walls of the prototype structure.*

9.6.4.1.3 *– Procedures shall be specified for designing and detailing the precast structural walls so that they have adequate ductility capacity. These procedures shall cover wall shear strength, sliding shear strength, boundary tie spacing to prevent bar buckling, concrete confinement, reinforcement strain, and any other actions or elements of the wall system that can affect ductility capacity.*

9.6.4.1.4 *– Procedures shall be specified for determining that an undesirable mechanism of nonlinear response, such* *as a story mechanism due to local buckling of the reinforcement or splice failure, or overall instability of the wall, does not occur.*

9.6.4.2 *– The design procedure shall be used to design the test modules and shall be documented in the test report.*

9.6.4.3 *– The design procedure used to proportion the test specimens shall define the mechanism by which the system resists gravity and earthquake effects, and shall establish acceptance values for sustaining this mechanism. Portions of the mechanism that deviate from code requirements shall be contained in the test specimens and shall be tested to determine acceptance values.*

9.6.5 – Test Modules:

9.6.5.1 *– At least two modules shall be tested. At least one module shall be tested for each limiting engineering design criterion (shear, axial load and flexure) for each characteristic configuration of precast structural walls, including intersecting structural walls or coupled structural walls. If all the precast walls of the structure have the same configuration and the same limiting engineering design criterion, then two modules shall be tested. Where intersecting precast wall systems are to be used, the response for the two orthogonal directions shall be tested.*

9.6.5.2 *– Where the design requires the use of coupling elements, those elements shall be included as part of the test module.*

9.6.5.3 *– Modules shall have a scale large enough to represent the complexities and behavior of the real materials and of the load transfer mechanisms in the prototype walls and their coupling elements, if any. Modules shall be of a scale not less than one half and shall be full-scale modules if the validation testing has not been preceded by an extensive analytical and experimental development program in which critical details of connections are tested at full scale.*

9.6.5.4 *– The geometry, reinforcing details, and materials properties of the walls, connections, and coupling elements shall be representative of those to be used in the prototype structure.*

9.6.5.5 *– Walls shall be at least two panels high unless the prototype structure is one for which a single panel is to be used for the full height of the wall.*

9.6.5.6 *– Where precast walls are to be used for bearing wall structures, as defined in ASCE/SEI 7-02,*[1.0] *the test modules shall be subject during lateral loading to an axial load stress representative of that anticipated at the base of the wall in the prototype structure.*

9.6.5.7 *– The geometry, reinforcement, and details used to connect the precast walls to the foundation shall replicate those to be used in the prototype structure.*

9.6.5.8 *– Foundations used to support the test modules shall have geometric characteristics, and shall be reinforced and supported, so that their deformations and cracking do not affect the performance of the modules in a way that would be different than in the prototype structure.*

9.6.6 – Testing Agency:

Testing shall be carried out by an independent testing agency approved by the Authority Having Jurisdiction. The testing agency shall perform its work under the supervision of a registered design professional experienced in seismic structural design.

9.6.7 – Test Method:

9.6.7.1 *– Test modules shall be subjected to a sequence of displacement-controlled cycles representative of the drifts expected under earthquake motions for the prototype structure. If the module consists of coupled walls, approximately equal drifts (within 5 percent of each other) shall be applied to the top of each wall and at each floor level. Cycles shall be to predetermined drift ratios as defined in Sections 9.6.7.2 through 9.6.7.6.*

9.6.7.2 *– Three fully reversed cycles shall be applied at each drift ratio.*

9.6.7.3 *– The initial drift ratio shall be within the essentially linear elastic response range for the module (see Section 9.6.7.11). Subsequent drift ratios shall be to values not less than 5/4 times, and not more than 3/2 times, the previous drift ratio.*

9.6.7.4 *– For uncoupled walls, testing shall continue with gradually increasing drift ratios until the drift ratio in percent equals or exceeds the larger of: (a) 1.5 times the drift ratio corresponding to the design displacement; or (b) the following value:*

$$0.80 \leq 0.67\,(h_w/l_w) + 0.5 \leq 2.5 \qquad (9.6.1)$$

where

h_w = *height of entire wall for prototype structure, in.*

l_w = *length of entire wall in direction of shear force, in.*

9.6.7.5 *– For coupled walls,* h_w/l_w *in Eq. (9.6.1) shall be taken as the smallest value of* h_w/l_w *for any individual wall of the prototype structure.*

9.6.7.6 *– Validation by testing to limiting drift ratios less than those given by Eq. (9.6.1) shall be acceptable provided testing is conducted in accordance with this document to drift ratios equal to or exceeding those determined for the response to a suite of nonlinear time history analyses conducted in accordance with Section 9.5.8 of ASCE/SEI 7-02*[1.0] *for maximum considered ground motions.*

9.6.7.7 *– Actual yield strength of steel reinforcement shall be obtained by testing coupons taken from the same reinforcement batch as used in the test module. Two tests, conforming to the ASTM specifications cited in Section 3.5 of ACI 318, shall be made for each reinforcement type and size.*

9.6.7.8 *– Actual compressive strength of concrete shall be determined by testing of concrete cylinders cured under the same conditions as the test module and tested at the time of testing the module. Testing shall conform to the applicable requirements of Sections 5.6.1 through 5.6.4 of ACI 318.*

9.6.7.9 *– Where strength and deformation capacity of coupling devices does not depend on reinforcement tested as required in Section 9.6.7.7, the effective yield strength and deformation capacity of coupling devices shall be obtained by testing independent of the module testing.*

9.6.7.10 *– Data shall be recorded from all tests such that a quantitative interpretation can be made of the performance of the modules. A continuous record shall be made of* *the test module drift ratio versus applied lateral force, and photographs shall be taken that show the condition of the test module at the peak displacement and after each key testing cycle.*

9.6.7.11 *– The effective initial stiffness of the test module shall be calculated based on test cycles to a force between $0.6E_{nt}$ and $0.9E_{nt}$, and using the deformation at the strength of $0.75E_{nt}$ to establish the stiffness.*

9.6.8. Test Report:

9.6.8.1 *– The test report shall contain sufficient evidence for an independent evaluation of all test procedures, design assumptions, and the performance of the test modules. As a minimum, all of the information required by Sections 9.6.8.1.1 through 9.6.8.1.11 shall be provided.*

9.6.8.1.1 *– A description shall be provided of the design procedure and theory used to predict test module strength, specifically the test module nominal lateral resistance, E_n, and the test module actual lateral resistance, E_{nt}.*

9.6.8.1.2 *– Details shall be provided of test module design and construction, including fully dimensioned engineering drawings that show all components of the test specimen.*

9.6.8.1.3 *– Details shall be provided of specified material properties used for design, and actual material properties obtained by testing in accordance with Section 9.6.7.7.*

9.6.8.1.4 *– A description shall be provided of test setup, including fully dimensioned diagrams and photographs.*

9.6.8.1.5 *– A description shall be provided of instrumentation, its locations, and its purpose.*

9.6.8.1.6 *– A description and graphical presentation shall be provided of applied drift ratio sequence.*

9.6.8.1.7 *– A description shall be provided of observed performance, including photographic documentation, of the condition of each test module at key drift ratios including (as applicable) the ratios corresponding to first flexural cracking or joint opening, first shear cracking, first crushing of the concrete for both positive and negative loading directions, and any other significant damage events that occur. Photos shall be taken at peak drifts and after the release of load.*

9.6.8.1.8 *– A graphical presentation shall be provided of the lateral force versus drift ratio response.*

9.6.8.1.9 *– A graphical presentation shall be provided of relative energy dissipation ratio versus drift ratio.*

9.6.8.1.10 *– A calculation shall be provided of effective initial stiffness for each test module, as observed in the test and as determined in accordance with Section 9.6.7.11, and a comparison made as to how accurately the design procedure has been able to predict the measured stiffness. The design procedure shall be used to predict the overall structural response and a comparison made as to how accurately this procedure has been able to predict the measured response.*

9.6.8.1.11 *– The test date, report date, name of testing agency, report author(s), supervising registered design professional, and test sponsor shall be provided.*

9.6.9 Test Module Acceptance Criteria:

9.6.9.1 *– The test module shall be deemed to have performed satisfactorily when all of the criteria for Sections 9.6.9.1.1 through 9.6.9.1.3 are met for both directions of in-plane response. If any test module fails to pass the validation testing required by these provisions for any test direction, then the wall system has failed the validation testing.*

9.6.9.1.1 *– Peak lateral strength obtained shall be at least $0.9E_{nt}$ and not greater than $1.2\ E_{nt}$.*

9.6.9.1.2 *– In cycling up to the drift level given by Sections 9.6.7.4 through 9.6.7.6, fracture of reinforcement or coupling elements, or other significant strength degradation, shall not occur. For a given direction, peak lateral strength during any cycle of testing to increasing displacement shall not be less than 0.8 times E_{max} for that direction.*

9.6.9.1.3 *– For cycling at the given drift level for which acceptance is sought in accordance with Sections 9.6.7.4, 9.6.7.5 or 9.6.7.6, as applicable, the parameters describing the third complete cycle shall have satisfied the following:*

1. The relative energy dissipation ratio shall have been not less than 1/8; and

2. The secant stiffness between drift ratios of –1/10 and +1/10 of the maximum applied drift shall have been not less than 0.10 times the stiffness for the initial drift ratio specified in Section 9.6.7.3.

9.6.10 – Reference:

1.0. ASCE/SEI, *ASCE/SEI Minimum Design Loads for Buildings and Other Structures – Earthquake Loads,* ASCE/SEI 7-02, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA, 2002.

COMMENTARY

9.6.1. – Notation:

Symbols additional to those in Chapter 21 of ACI 318 are defined:

A_h = *area of hysteresis loop*

E_1, E_2 = *peak lateral resistance for positive and negative loading, respectively, for third cycle of loading sequence*

f_1 = *live load factor defined in Section 9.6.2.6*

h_w = *height of column of test module, in. or mm*

K, K' = *initial stiffness for positive and negative loading, respectively, for first cycle*

θ_1, θ_2 = *drift ratios at peak lateral resistance for positive and negative loading, respectively, for third cycle of loading sequence*

θ_1', θ_2' = *drift ratios for zero lateral load for unloading at stiffnesses K, K' from peak positive and negative lateral resistance, respectively, for third cycle of loading sequence (Fig. C9.6.2.5)*

Δ = *lateral displacement, in. or mm (see Figs. C9.6.2.2.1, C9.6.2.2.2 and C9.6.2.2.3)*

Δ_a = *allowable story drift, in. or mm (see Table 9.5.2.8 of SEI/ASCE 7-02)*

9.6.2 – Definitions:

9.6.2.1 – Coupling elements *– Coupling elements are connections provided at specific intervals along the vertical boundaries of adjacent structural walls. Coupled structural walls are stiffer and stronger than the same walls acting in-*

dependently. For cast-in-place construction, effective coupling elements are typically coupling beams having small span-to-depth ratios. The inelastic behavior of such beams is normally controlled by their shear strength.

For precast construction, effective coupling elements can be precast beams connected to the adjacent structural walls either by post-tensioning, ductile mechanical devices, or grouted-in-place reinforcing bars.[2] The resultant coupled construction can be either emulative of cast-in-place construction or non-emulative (jointed). However, for precast construction, coupling beams can also be omitted and mechanical devices used to connect directly the vertical boundaries of adjacent structural walls.[2,3]

***9.6.2.2 – Drift ratio** – The definition of the drift ratio for a three-panel wall module, θ, is illustrated in Fig. C9.6.2.2.1. The position of the module at the start of testing, with only its self-weight acting, is indicated by broken lines. The module is set on a horizontal foundation support that is centered at A and is acted on by a lateral force, H, applied at the top of the wall. The self-weight of the wall is distributed uniformly to the foundation support.*

However, under lateral loading, this self-weight and any axial gravity load acting at the top of the wall cause overturning moments on the wall that are additional to the overturning moment, Hh_w, and can affect deformations. The chord, AB, of the centroidal axis of the wall is the vertical reference line for drift measurements.

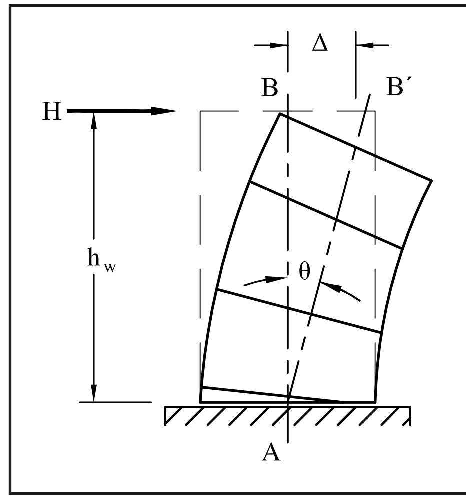


Fig. C9.6.2.2.1. Definition of drift ratio *θ*.

For acceptance testing, a lateral force, H, is applied to the wall through the pin at B. Depending on the geometric and reinforcement characteristics of the module, this force can result in the module taking up any one, or a combination, of the deformed shapes indicated by solid lines in Figs. C9.6.2.2.1, C9.6.2.2.2 and C9.6.2.2.3. Figure C9.6.2.2.2 il-

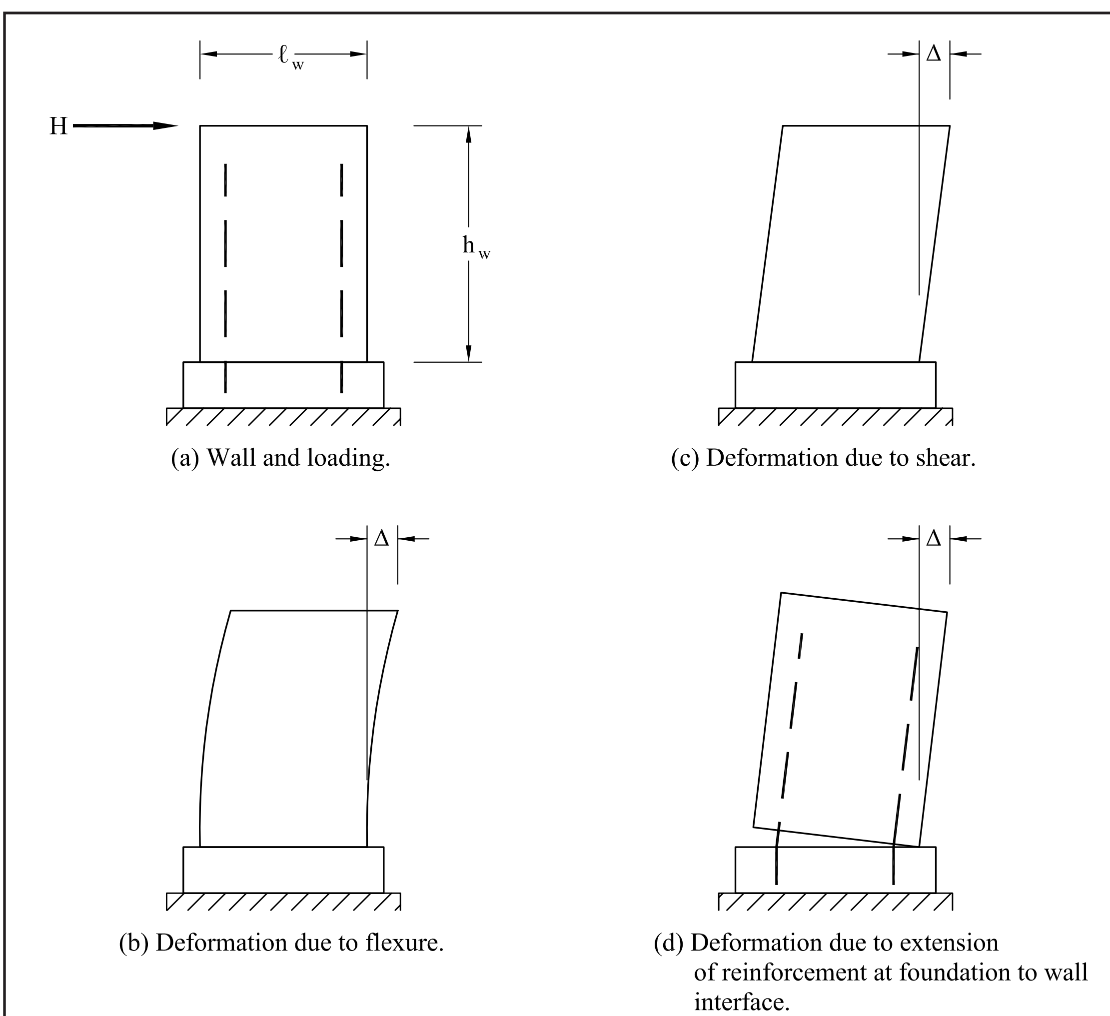


Fig. C9.6.2.2.2. Typical wall deformation components.

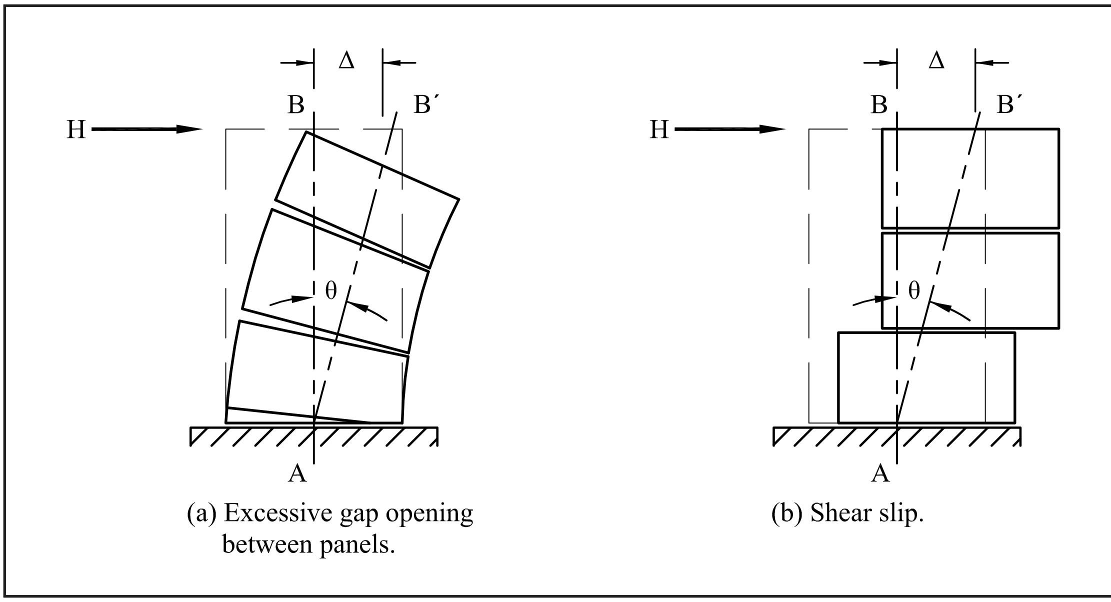


Fig. C9.6.2.2.3. Undesirable deformations along horizontal joints: (a) excessive gap opening between panels; (b) shear slip.

lustrates several possible components of the displacement, Δ, for a wall that is effectively solid, while Fig. C9.6.2.2.3 illustrates two possibly undesirable components of the displacement Δ.

Regardless of the mode of deformation of the wall, the lateral force causes the wall at B to displace horizontally by an amount Δ. The drift ratio is the angular rotation of the wall chord with respect to the vertical, and for the setup shown, equals Δ / h_w, where h_w is the wall height and is equal to the distance between the foundation support at A and the load point at B.

Where prestressing steel is used in wall members, the stress, f_{ps}, in the reinforcement at the nominal and the probable lateral resistance shall be calculated in accordance with Section 18.7 of ACI 318.

***9.6.2.3** – **Global toughness** – These provisions describe acceptance criteria for special precast structural walls based on validation testing. The requirements of Section 21.2.1.5 of ACI 318 concerning toughness cover both the energy dissipation of the wall system which, for monolithic construction, is affected primarily by local plastic hinging behavior and the toughness of the prototype structure as a whole. The latter is termed "global toughness" in these provisions and is a condition that does not apply to the walls alone. That global toughness requirement can be satisfied only though analysis of the performance of the prototype structure as a whole when the walls perform to the criteria specified in these provisions.*

The required gravity load for global toughness evaluations is the value given by these provisions. For conformity with Section 9.2.1 of ACI 318, UBC 1997,[7] IBC 2003[11] and NFPA 5000[12] the required gravity load is $1.2D + f_1L$, where the seismic force is additive to gravity forces and 0.9D, where the seismic force counteracts gravity forces. D is the effect of dead loads, L is the effect of live loads, and f_1 is a factor equal to 0.5 except for parking structures, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf (4.79 kN/m^2), where f_1 equals 1.0.

***9.6.2.5** – **Relative energy dissipation ratio** – This concept is illustrated in Fig. C9.6.2.5 for the third loading cycle to the limiting drift ratio required by Sections 9.6.7.4, 9.6.7.5 or 9.6.7.6, as appropriate. For Fig. C9.6.2.5, it is assumed that the test module has exhibited different initial stiffnesses, K and K′, for positive and negative lateral forces and that the peak lateral resistances for the third cycle for the positive and negative loading directions, E_1 and E_2, also differ.*

The area of the hysteresis loop in Fig. C9.6.2.5 for the third cycle, A_h, is shown with cross-hatched lines. The circumscribing figure consists of two parallelograms, ABCD and DFGA. The slopes of the lines AB and DC are the same as the initial stiffness, K, for positive loading, and the slopes of the lines DF and GA are the same as the initial stiffness, K′, for negative loading. The relative energy dissipation ratio concept is similar to the equivalent viscous damping concept used in Section 13.9.3 of the 2000 NEHRP Provisions and Commentary[1] for required tests of seismic isolation systems.

For a given cycle, the relative energy dissipation ratio, β, is the area, A_h, inside the lateral force-drift ratio loop for the module, divided by the area of the effective circumscribing parallelograms ABCD and DFGA. The areas of the parallelograms equal the sum of the absolute values of the lateral force strengths, E_1 and E_2, at the drift ratios θ_1 and θ_2 multiplied by the sum of the absolute values for the drift ratios θ'_1 and θ'_2.

9.6.3.** – **Scope and General Requirements:

While only ACI Committee 318 can determine the requirements necessary for precast walls to meet the provisions of Section 21.2.1.5 of ACI 318, Section 1.4 of ACI 318 already permits the building official to accept wall systems, other than those explicitly covered by Chapter 21 of ACI 318, 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. The purpose of these provisions is to provide a framework that establishes the specific tests, load

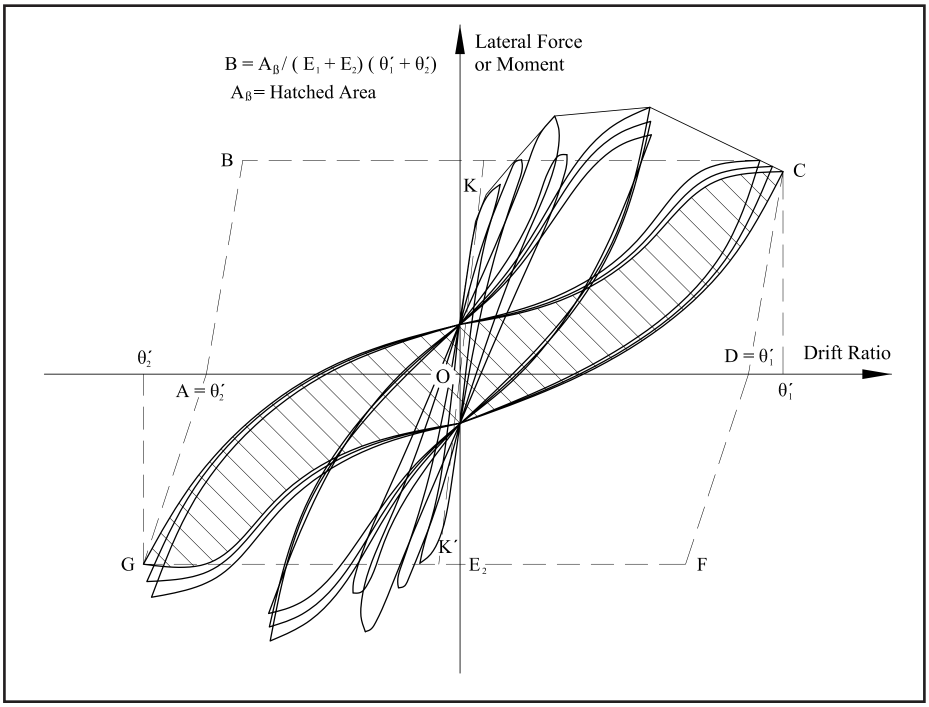


Fig. C9.6.2.5. Relative energy dissipation ratio.

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 precast wall systems, including coupled wall systems, not satisfying all the requirements of Chapter 21 of ACI 318. For regions of moderate seismic risk or for structures assigned to intermediate seismic performance or design categories, less stringent provisions than those specified here are appropriate.

These provisions assume that the precast wall system to be tested has details differing from those prescribed by Section 21.7 of ACI 318 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.

For monolithic reinforced concrete walls, a fundamental design requirement of Chapter 21 of ACI 318 is that walls with h_w/l_w *exceeding1.0 be proportioned so that their inelastic response is dominated by flexural action on a critical section located near the base of the wall. That fundamental requirement is retained in these provisions. The reason is that tests on modules, as envisioned in these provisions, 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 318 if the shear-slip displacement pattern of Fig. C9.6.2.2.3, or the shear deformation response of Fig. C9.6.2.2.2, governs the response developed in the test on the module. Two other fundamental requirements of Chapter 21 of ACI 318 are for 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 is expected to exceed some critical value. 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 drift ratios specified in Sections 9.6.7.4, 9.6.7.5 and 9.6.7.6. 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.*

***9.6.3.1** – These provisions are not intended for use with existing construction or for use with walls that are designed to conform to all the requirements of Section 21.7 of ACI 318. The criteria of these provisions are more stringent than those for walls designed to Section 21.7 of ACI 318. Some walls designed to Section 21.7, and having low height-to-length ratios, may not meet the drift ratio limits of Eq. 9.6.1 because their behavior may be governed by shear deformations.[13] The height-to-length ratio of 0.5 is the least value for which Eq. 9.6.1 is applicable.*

***9.6.3.3** - For acceptance, the results of the tests on each module must satisfy the acceptance criteria of Section 9.6.9. In particular, the relative energy dissipation ratio calculated from the measured results for the third cycle between the specified limiting drift ratios must equal or exceed 1/8. For uncoupled walls, relative energy dissipation ratios increase as the drift ratio increases.[4] Tests on slender monolithic walls have shown relative energy dissipation ratios, derived from rotations at the base of the wall, of about 40 to 45 percent at large drifts.[5] The same result has been reported even where there has been a significant opening in the web of the wall on the compression side.[6] For 0.020 drift ratios and walls with height-to-length ratios of 4, relative energy dissipation ratios have been computed[4] as 30, 18, 12, and 6 percent, 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 percent of the flexural strength provided by deformed bar reinforcement and 75 percent by the prestressed reinforcement, and post-tensioned prestressed concrete special structural walls, respectively. Thus, for slender precast uncoupled walls of emulative or non-emulative design, it is to be anticipated that at least 35 percent of the flexural capacity at the base of the wall needs to be provided by deformed bar reinforcement if the requirement of a relative energy dissipation ratio of 1/8 is to be*

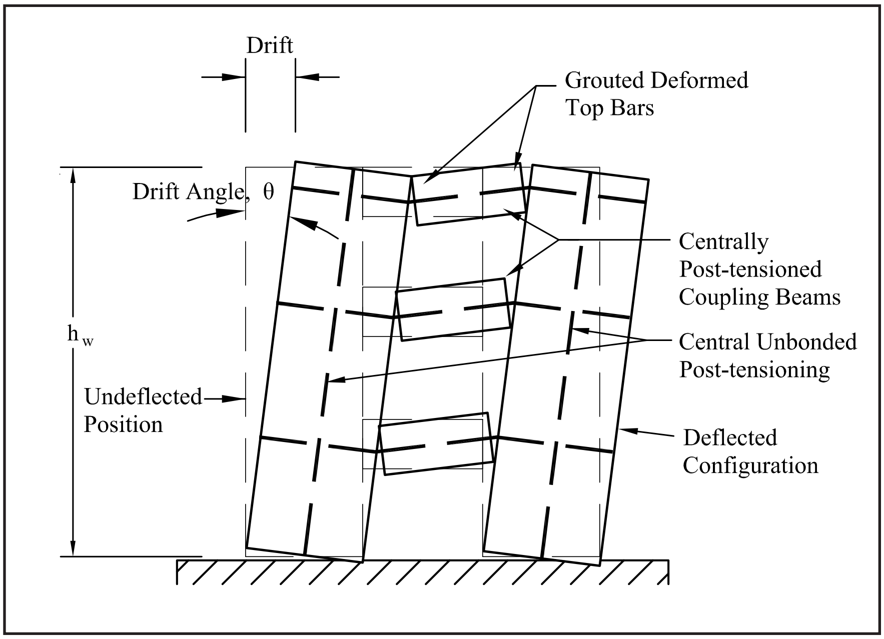


Fig. C9.6.5.1(a). Coupled wall test module.

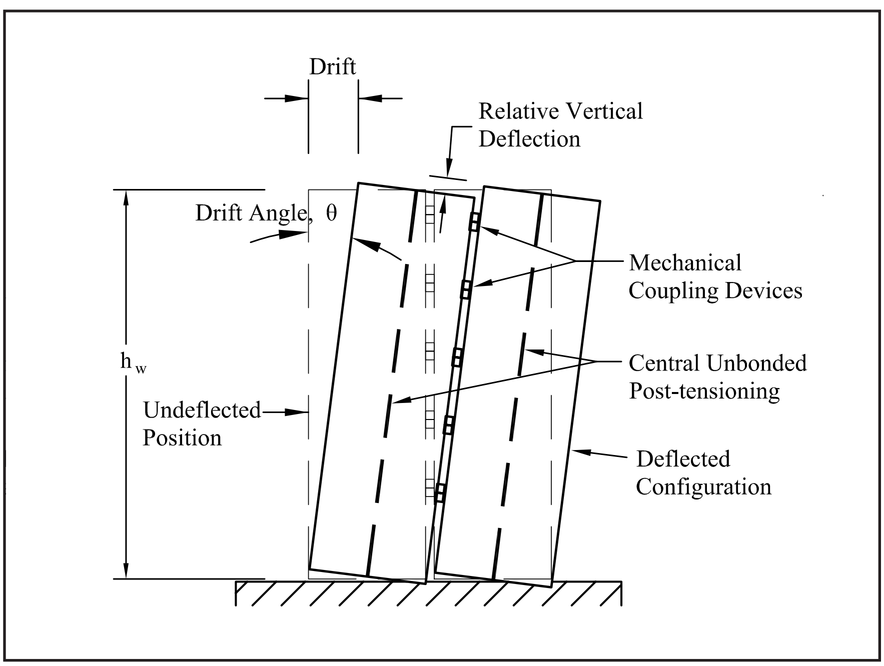


Fig. C9.6.5.1(b). Coupled wall test module with vertical mechanical couplers.

achieved. However, if more than about 40 percent of the flexural capacity at the base of the wall is provided by deformed bar reinforcement, then the self-centering capability of the wall following a major event is lost and that is one of the prime advantages gained with the use of post-tensioning. For squat walls with height-to-length ratios between 0.35 and 0.69, the relative energy dissipation has been reported[13] as remaining constant at 23 percent for drifts between that for first diagonal cracking and that for a post-peak capacity of 80 percent 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 coupled wall systems, theoretical studies[14] and tests[9] 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.

***9.6.3.3.4** - The SEI/ASCE 7-02 allowable story drift limits are the basis for the drift limits of IBC 2003 and NFPA 5000. Allowable story drifts, Δ_a, are specified in Table 1617.3 of IBC 2003 and likely values are discussed in the Commentary to Section 9.6.7.4. The limiting initial drift ratio consistent with Δ_a equals $\Delta_a/\Phi C_d h_w$, 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 Δ_a/h_w equal to 0.015, the required deflection amplification factor C_d of 5, and Φ equal to 0.9, the limiting initial drift ratio, corresponding to B in Fig. C9.6.9.1, 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, **E**, while the limiting initial drift ratio is at the nominal strength, E_n, which must be greater than E/Φ. The load-deformation relationship of a wall becomes significantly nonlinear before the applied load reaches 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 about 75 percent of E_{nt}.*

***9.6.3.3.5** - The criteria of Section 9.6.9 are for the test module. In contrast, the criterion of Section 9.6.3.3.5 is for the structural system as a whole, and can be satisfied only by the philosophy used for the design and analysis of the building as a whole. The criterion adopted here is similar to that described in the last paragraph of R21.2.1 of ACI 318 and the intent is that test results and analyses demonstrate that the structure, after cycling three times through both positive and negative values of the limiting drift ratio specified in Sections 9.6.7.4, 9.6.7.5 or 9.6.7.6, as appropriate, is still capable of supporting the gravity load specified as acting on it during the earthquake.*

9.6.4 - Design Procedure:

***9.6.4.1** - The test program specified in these provisions is intended to verify an existing design procedure for precast structural walls for a specific structure or for prequalifying a generic type of special precast wall system for construc-*

tion in general. The test program is not for the purpose of creating basic information on the strength and deformation properties of such systems for design purposes. Thus, the test modules should not fail during the validation testing, a result that is the opposite of what is usually necessary during testing in the development phase for a new or revised design procedure. For a generic precast wall system to be accepted based on these provisions, a rational design procedure is to have been developed prior to this validation testing. The design procedure is to be based on a rational consideration of material properties and force transfer mechanisms, and its development will usually require preliminary and possibly extensive physical testing that is not part of the validation testing. Because special wall systems are likely to respond inelastically during design-level ground shaking, the design procedure must consider wall configuration, equilibrium of forces, compatibility of deformations, the magnitudes of the lateral drifts, reversed cyclic displacements, the relative values of each limiting engineering design criteria (shear, flexure and axial load) and use appropriate constitutive laws for materials that include considerations of effects of cracking, loading reversals and inelasticity.

The effective initial stiffness of the structural walls is important for calculating the fundamental period of the prototype structure. The procedure used to determine the effective initial stiffness of the walls is to be verified from the validation test results as described in Section 9.6.7.11.

Provisions 9.6.4.1.1 through 9.6.4.1.3 state the minimum procedures to be specified in the design procedure prior to the start of testing. The Authority Having Jurisdiction may require that more details be provided in the design procedure than those of Sections 9.6.4.1.1 through 9.6.4.1.3 prior to the start of testing.

***9.6.4.2** - The justification for the small number of test modules, specified in Section 9.6.5.1 is that a previously developed rational design procedure is being validated by the test results. Thus, the test modules for the experimental program must be designed using the procedure intended for the prototype wall system and strengths must be predicted for the test modules before the validation testing is started.*

9.6.5 - Test Modules:

***9.6.5.1** – One module must be tested for each limiting engineering design criterion, such as shear, or axial load and flexure, for each characteristic configuration of walls. Thus, in accordance with Section 9.6.4.3, if the test on the module results in a maximum shear stress of $3\sqrt{f_c'}$, then the maximum shear stress that can be used in the prototype is that same value. Each characteristic in-plane configuration of walls, or coupled walls, in the prototype structure must also be tested. Thus, as a minimum for one-way structural walls, two modules with the configuration shown in Fig. C9.6.2.2.1 must be tested, and for one-way coupled walls, two modules with the configuration shown in either Fig. C9.6.5.1(a) or in Fig. C9.6.5.1(b) must be tested. In addition, if intersecting wall systems are to be used, then the response of the wall systems for the two orthogonal directions needs to be tested. For two-way wall systems and coupled wall-frame systems, testing of configurations other than those shown in Figs. C9.6.2.2.1 and C9.6.5.1 may be appropriate when it is difficult to realistically model the likely dominant earthquake deformations using orthogonal direction testing only.*

This provision should not be interpreted as implying that only two tests will need to be made to qualify a generic system. During the development of that system, it is likely that several more tests will have been made, resulting in progressive refinements of the mathematical model used to describe the likely performance of the generic structural wall system and its construction details. Consequently, only one test of each module type for each limiting engineering design condition, at a specified minimum scale and subjected to specific loading actions, may be required to validate the system. Further, as stated in Section 9.6.9.1, if any one of those modules for the generic wall system fails to pass the validation testing required by these provisions, then the generic wall system has failed the validation testing.

In most prototype structures, a slab is usually attached to the wall and, as demonstrated by the results of the PRESSS building test,[9] the manner in which the slab is connected to the wall needs to be carefully considered. The connection needs to be adequate to allow the development of story drifts equal to those anticipated in these provisions. However, in conformity with common practice for the sub-assemblage tests used to develop the provisions of Chapter 21 of ACI 318, there is no requirement for a slab to be attached to the wall of the test module. The effect of the presence of the slab should be examined in the development program that precedes the validation testing.

***9.6.5.3** - Test modules need not be as large as the corresponding walls in the prototype structure. The scale of the test modules, however, must be large enough to capture all the complexities associated with the materials of the prototype wall, its geometry and reinforcing details, load transfer mechanisms, and joint locations. For modules involving the use of precast elements, for example, scale effects for load transfer through mechanical connections should be of particular concern.[3] The issue of the scale necessary to capture fully the effects of details on the behavior of the prototype should be examined in the development program that precedes the validation testing.[15]*

***9.6.5.4** - It is to be expected that for a given generic precast wall structure, such as an unbonded centrally post-tensioned wall constructed using multiple precast or precast pretensioned concrete wall panels, validation testing programs will initially use specific values for the specified strength of the concrete and reinforcement in the walls, the layout of the connections between panels, the location of the post-tensioning, the location of the panel joints, and the design stresses in the wall. Pending the development of an industry standard for the design of such walls, similar to the standard for special hybrid moment frames,[16] specified concrete strengths, connection layouts, post-tensioning amounts and locations, etc., used for such walls will need to be limited to the values and layouts used in the validation testing programs.*

***9.6.5.5** – For walls constructed using precast or precast/prestressed panels and designed using non-emulative methods, the response under lateral load can change significantly with the joint opening [Fig. C9.6.2.2.2(d) and Fig.*

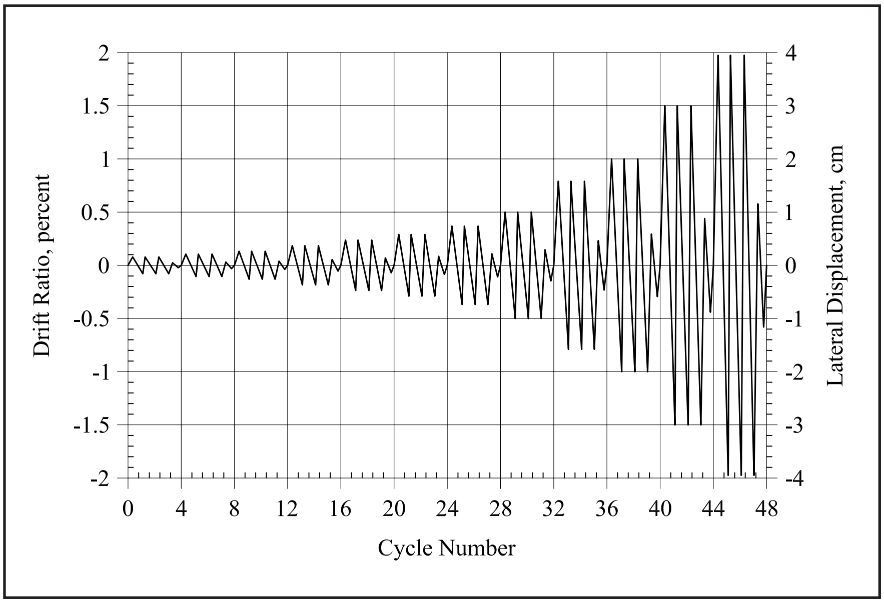


Fig. C9.6.7. Example of specified test sequence.

C9.6.2.2.3(a)]. The number of panels used to construct a wall depends on wall height and design philosophy. If, in the prototype structure, there is a possibility of horizontal joint opening under lateral loading at a location other than the base of the wall, then the consequences of that possibility need to be considered in the development and validation test programs. Joint opening at locations other than the base can be prevented through the use of capacity design procedures.

9.6.5.6 - *The significance of the magnitude of the gravity load that acts simultaneously with the lateral load needs to be addressed during the validation testing if the development program suggests that effect is significant.*

9.6.5.7 – *Details of the connection of walls to the foundation are critical, particularly for non-emulative wall designs. The deformations that occur at the base of the wall due to plastic hinging or extension of the reinforcing bars or post-tensioning steel crossing the wall to foundation interface [Fig. C9.6.2.2.2(d)] are in part determined by details of the anchorage and the bonding of those reinforcements on either side of the interface. Grout will be normally used to bed panels on the foundation and the characteristics of that grout in terms of materials, strength and thickness can have a large effect on wall performance. The typical grout pad with a thickness of 1 in. (25 mm) or less can be expected to provide a coefficient of friction of about 0.6 under reversed loadings.*[17,18] *Pads with greater thickness and without fiber reinforcement exhibit lesser coefficients of friction. Adequate frictional resistance is essential to preventing undesirable shear-slip deformations of the type shown in Fig. C9.6.2.2.3(b).*

9.6.5.8 *- The geometry of the foundations need not duplicate that used in the prototype structure. However, the geometric characteristics of the foundations (width, depth and length) need to be large enough that they do not influence the behavior of the test module.*

9.6.6 - Testing agency:

In accordance with the spirit of the requirements of Sections 1.3.5 and 1.4 of ACI 318, it is important that testing be carried out by a recognized independent testing agency, approved by the Authority Having Jurisdiction and that the testing and reporting be supervised by a registered design professional familiar with the proposed design procedure and experienced in testing and seismic structural design.

9.6.7 - Test Method:

The test sequence is expressed in terms of drift ratio, and the initial ratio is related to the likely range of linear elastic response for the module. That approach, rather than testing at specific drift ratios of 0.005, 0.010, etc., is specified because, for modules involving prestressed concrete, the likely range of elastic behavior varies with the prestress level.[14,15]

An example of the test sequence specified in Sections 9.6.7.2 through 9.6.7.6 is illustrated in Fig. C9.6.7. The sequence is intended to ensure that displacements are increased gradually in steps that are neither too large nor too small. If the steps are too large, the drift capacity of the system may not be determined with sufficient accuracy. If the steps are too small, the system may be unrealistically softened by loading repetitions, resulting in artificially low maximum lateral resistances and artificially high maximum drifts. Also, when the steps are too small, the rate of change of energy stored in the system may be too small compared with the change occurring during a major event. Results, using such small steps, can mask undesirable brittle failure modes that might occur in the inelastic response range during a major event. Because significant diagonal cracking is to be expected in the inelastic range in the web of walls, and in particular in squat walls, the pattern of increasing drifts used in the test sequence can markedly affect diagonal crack response in the post-peak range of behavior.[13]

The drift capacity of a building in a major event is not a single quantity, but depends on how that event shakes the structure. In the forward near field, a single pulse may determine the maximum drift demand, in which case a single large drift demand cycle for the test module would give the best estimation of the drift capacity. More often, however, many small cycles precede the main shock and that is the scenario represented by the specified loading.

There is no requirement for an axial load to be applied to the wall simultaneously with the application of the lateral displacements. In many cases, it will be conservative not to apply an axial load because, in general, the shear capacity of the wall and the resistance to slip at the base of the wall increase as the axial load on the wall increases. However, as the height of the wall increases and the limiting drift utilized in the design of the wall increases, the likelihood of extreme fiber crushing in compression at maximum drift increases, and the importance of the level of axial load *increases. The significance of the level of axial loading should be examined during the development phase.*

9.6.7.4 *- For the response of a structure to the design seismic shear force, current building codes such as UBC-97,[7] IBC 2003[11] or NFPA 5000,[12] or recommended provisions such as NEHRP-2000,[1] SEI/ASCE 7-02[2.0] and FEMA 273[19] specify a maximum allowable drift. However, structures designed to meet that drift limit may experience greater drifts during an earthquake equal to the design basis earthquake and are likely to experience greater drifts during an earthquake equal to the maximum credible earthquake. In addition to the characteristics of the ground motion, actual drifts will depend on the strength of the structure, its initial elastic stiffness, and the ductility expected for the given lateral load resisting system. Specification of suitable limiting drifts for the test modules requires interpretation and allowance for uncertainties in the assumed ground motions and structural properties.*

In IBC 2003, the design seismic shear force applied at the base of a building is related directly to its weight and the design elastic response acceleration, and inversely to a response modification factor, R. That R factor increases with the expected ductility of the lateral force resisting system of the building. Special structural walls satisfying the requirements of Sections 21.2 and 21.7 are assigned an R value of 6 when used in a building frame system and a value of 5 when used in a bearing wall system. They are also assigned allowable story drift ratios that are dependent on the hazard to which the building is exposed. When the design seismic shear force is applied to a building, the building responds inelastically and the resultant computed drifts (the design story drifts) must be less than a specified allowable drift. Additional guidance is given in FEMA 356,[19] where the deformations for rectangular walls with height-to-length ratios greater than 2.5, and flanged wall sections with height-to-length ratios greater than 3.5, are to be assumed to be controlled by flexural actions. When structural walls are part of a building representing a substantial hazard to human life in the event of a failure, the allowable story drift ratio for shear controlled walls is 0.0075 and for flexure controlled walls is a function of the plastic hinge rotation at the base of the wall. For flexure controlled walls, values range up to a maximum of about 0.02 for walls with confined boundary elements with low reinforcement ratios and shear stress less than $3\sqrt{f_c'}$.

To compensate for the use of the R value, IBC 1617.4.6 requires that the drift determined by an elastic analysis for the code-prescribed seismic forces be multiplied by a deflection amplification factor, C_d, *to determine the design story drift and that the design story drift must be less than the allowable story drift. In building frame systems, structural walls satisfying the requirements of Section 21.7 of ACI 318, are assigned a* C_d *value of 5. However, research[8] has found that design story drift ratios determined in the foregoing manner may be too low. Drift ratios of six times IBC-calculated values (rather than five) are more representative of the upper bounds to expected drift ratios. The value of six is also in agreement with the finding that the drift ratio of an inelastic structure is approximately the same as that of an elastic structure with the same initial period. For flexure controlled walls, the value of 6/5 times the present IBC limits on the calculated drift ratio, would lead to a limit on real drift ratios of up to 0.024.*

Duffy et al.[20] reviewed experimental data for shear walls to define post-peak behavior and limiting drift ratios for walls with height-to-length ratios between 0.25 and 3.5. Seo et al.[10] re-analyzed the data of Duffy et al. together with data from tests conducted subsequent to the analysis of Duffy et al. Duffy et al. established that for squat walls with web reinforcement satisfying ACI 318-02 requirements and height-to-length ratios between 0.25 and 1.1, there was a significant range of behavior for which drifts were still reliable in the post-peak response region. Typically, the post-peak drift increased by 0.005 for a 20 percent degradation in capacity under cyclic loading. For greater values of degradation, drifts were less reliable. That finding has also been confirmed through tests conducted by Hidalgo et al.[13] on squat walls with effective height-to-length ratios ranging between 0.35 and 1.0. Values of the drift ratio of the walls at inclined cracking and at peak capacity varied little with web reinforcement. By contrast, drifts in the post-peak range were reliable to a capacity equal to 80 percent of the peak capacity and were 0.005 greater than the drifts at peak capacity provided the walls contained horizontal and vertical web reinforcement equal to 0.25 percent.

From an analysis of the available test data, and from theoretical considerations for a wall rotating flexurally about a plastic hinge at its base, Seo et al.[10] concluded that the limiting drift at peak capacity increased almost linearly with the height-to-length ratio of the wall. When the additional post peak drift capacity for walls with adequate web reinforcement was added to the drift at peak capacity, then the total available drift capacity in percent was given by the following equation:

$$1.0 \le 0.67(h_w/l_w) + 0.5 \le 3.0$$

where h_w *is the height of the wall, and* l_w *is the length of the wall. The data from the tests of Hidalgo et al.[13] suggest that while that formula is correct for squat walls the lower limit on drift can be decreased to 0.8 as specified in these provisions and that the use of that formula should be limited to walls with height-to-length ratios equal to or greater than 0.5. For wall height-to-length ratios less than 0.5 the behavior is controlled principally by shear deformations [Fig. C9.6.2.2.2(c)] and Eq. 9.6.1 should not be used. The upper value of 0.030 for the drift ratio was somewhat optimistic because the data were for walls with height-to-length ratios equal to or less than 3.5 and subsequent tests[5,6] have shown that the upper limit of 2.5, as specified in Eq. 9.6.1, is a more realistic limit.*

9.6.7.5 – *The design capacity for coupled wall systems must be developed by the drift ratio corresponding to that for the wall with the least* h_w/l_w *value. However, it is desirable that testing be continued to the drift given by Eq. 9.6.1 for the wall with the greatest* h_w/l_w *ratio in order to assess the reserve capacity of the coupled wall system.*

9.6.7.6 *– The drift limits of Eq. 9.6.1 are representative of*

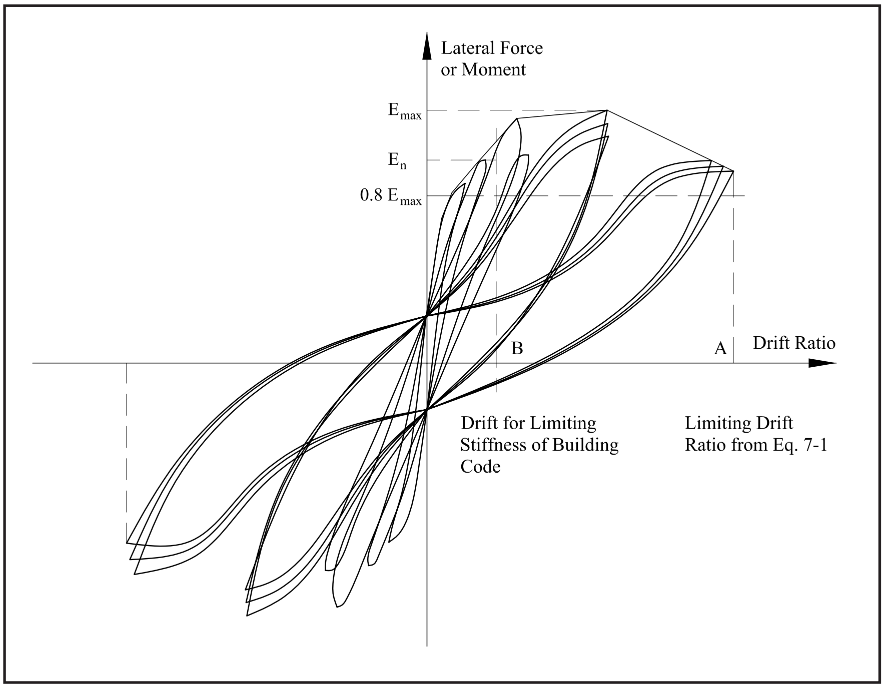


Fig. C9.6.9.1. Quantities used in evaluating acceptance criteria.

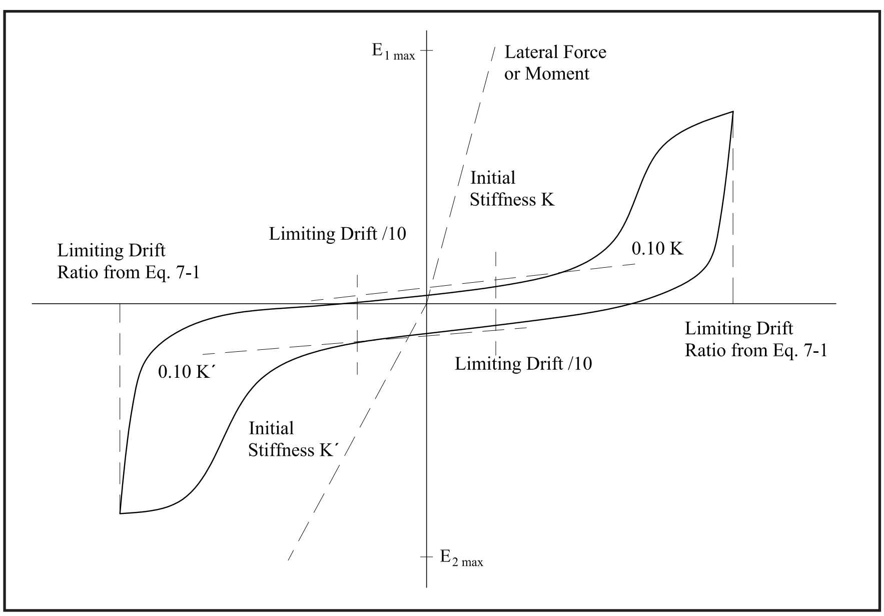


Fig. C9.6.9.1.3. Unacceptable hysteretic behavior.

the maximum that can be achieved by walls designed to ACI 318. The use of smaller drift limits is appropriate if the designer wishes to use performance measures less than the maximum permitted by ACI 318. Examples are the use of reduced shear stresses so that the likelihood of diagonal cracking of the wall is minimized or reduced compressive stresses in the boundary elements of the wall so that the risk of crushing is reduced. Nonlinear time history analyses for the response to a suite of maximum considered earthquake (MCE) ground motions, rather than 1.5 times a suite of the corresponding design basis earthquake (DBE) ground motions, is required because the drifts for the response to the MCE motion can be significantly larger than 1.5 times the drifts for the response to the DBE motions.

***9.6.7.10** - In many cases, data additional to the minimum specified in Section 9.6.7.7 may be useful to confirm both design assumptions and satisfactory response. Such data include relative displacements, rotations, curvatures, and strains.*

9.6.8 - Test Report:

The test report must be sufficiently complete and self-contained for a qualified expert to be satisfied that the tests have been designed and carried out in accordance with these criteria, and that the results satisfy the intent of these provisions. Provisions 9.6.8.1.1 through 9.6.8.1.11 state the minimum evidence to be contained within the test report. The Authority Having Jurisdiction or the registered design professional supervising the testing may require that additional test information be reported.

9.6.9 – Test module acceptance criteria:

The requirements of this clause apply to each module of the test program and not to an average of the results of the program. Fig. C9.6.9.1 illustrates the intent of this clause.

***9.6.9.1.1 -** Where nominal strengths for opposite loading directions differ, as is likely for C-, L- or T-shaped walls, the criterion of Section 9.6.9.1.1 applies separately to each direction.*

***9.6.9.1.2 -** At high cyclic-drift ratios, strength degradation is inevitable. To limit the level of degradation so that drift ratio demands do not exceed anticipated levels, a maximum strength degradation of **$0.20E_{max}$** is specified. Where strengths differ for opposite loading directions, this requirement applies independently to each direction.*

***9.6.9.1.3 –** (1) If the relative energy dissipation ratio is less than 1/8, there may be inadequate damping for the building as a whole. Oscillations may continue for some time after an earthquake, producing low-cycle fatigue effects, and displacements may become excessive.*

***9.6.9.1.3 –** (2) If the stiffness becomes too small around zero drift ratio, the structure will be prone to large displacements for small lateral force changes following a major earthquake. A hysteresis loop for the third cycle between peak drift ratios of 1/10 times the limiting drift ratio given by Eq. 9.6.1, that has the form shown in Fig. C9.6.9.1, is acceptable. At zero drift ratio, the stiffnesses for positive and negative loading are about 11 percent of the initial stiff-*

nesses. Those values satisfy Section 9.6.9.1.3. An unacceptable hysteresis loop form would be that shown in Fig. C9.6.9.1.3 where the stiffness around zero drift ratio is unacceptably small for both positive and negative loading.

9.6.10 - References

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FUTURE COURSE

The NEHRP provision permitting the use of non-emulative precast concrete walls, including the acceptance criteria based on validation testing, was proposed for inclusion in ASCE 7-05,[10] which will form the basis of the seismic design provisions of the 2006 International Building Code (IBC). The ASCE 7 Seismic Task Committee determined that while it was entirely consistent with BSSC's role to include Acceptance Criteria for Special Precast Structural Walls in the NEHRP Provisions, major modifications to ACI 318, such as the Acceptance Criteria, should not appear in ASCE 7; they should instead be processed by ACI Committee 318.

PCI realized that if the path that led to the inclusion of non-emulative special moment frames in ACI 318-02 were to be followed, an Innovation Task Group (ITG) would have to be formed within ACI to develop a provisional standard similar to ACI T1.1-01 for precast shear wall systems. Such an ITG (ITG 5) has in fact been formed at the request of PCI. The scope has narrowed further as a result of deliberations that have taken place so far within the ITG. The specific technology for which that ITG is developing documents is the use of unbonded post-tensioned special precast concrete structural walls that are connected along their vertical edges with energy dissipating mechanical couplers.

To introduce the technology, the ITG envisages that, similar to the process used for ITG 1, two standards will need to be developed. One would be on "Acceptance Criteria for Special Unbonded Post-Tensioned Structural Walls Based on Validation Testing." That standard would detail the experimental evidence and analysis required (ACI 318-02 Section 21.2.1.5) to allow the use of special structural walls not satisfying the prescriptive requirements of Section 21.7 of ACI 318-02. The second standard would detail design requirements for "Unbonded Post-Tensioned Special Precast Structural Walls," and would cover both isolated walls and walls connected along their vertical boundaries with energy dissipating mechanical couplers. Those walls would differ from the emulative walls specified in Section 21.8 of ACI 318-02.

If all goes well, a provisional standard similar to T1.1-01 may be approved by the Standards Board of ACI by the fall of 2005 (this is the most optimistic scenario). If that transpires, it should be possible to have provisions included in ACI 318-08, which would permit non-emulative design of special precast shear walls using the provisional standard. If ACI 318-08 is missed, which can happen relatively easily, the provisions should make it into ACI 318-11. ACI 318-08 will be the reference document for the 2009 IBC and ACI 318-11 will be the reference document for the 2012 IBC.

Meanwhile, an enterprising developer who is prepared to get validation testing done, as required by the above acceptance criteria, may seek building department approval of individual projects, citing the NEHRP provision, under Section 104.11of the 2003 IBC, which states:

104.11 Alternative materials, design and methods of construction and equipment. *The Provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method, or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.*

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