

Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary

An ACI Standard

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This document defines the minimum experimental evidence that can be deemed adequate to attempt to validate the use, in regions of high seismic risk or in structures assigned to satisfy high seismic performance or design categories, of weak beam/strong column moment frames not satisfying fully the prescriptive requirements of Chapter 21 of ACI 318-99. This document consists of both a Standard and a Commentary that is not part of the Standard.

ACI Committee 374 adopted ACI T1.1/T1.1R-01 as ACI 374.1-05 on October 12, 2005. ACI T1.1/T1.1R-01 superseded ACI ITG/T1.1-99 and became effective March 9, 2001. Copyright © 2005, American Concrete Institute.

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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 318-99 and the corresponding sections of ACI 318R-99. Among the subjects covered are requirements for: procedures that shall be used to design test modules; configurations for those modules; test methods; test reports; and determination of satisfactory performance.

The Commentary describes some of the considerations of the Innovation Task Group in developing the Standard. 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.

The Commentary references documentary evidence, additional to the references of Chapter 21 of ACI 318R-99, that supports the Standard. Consistent with the approach of ACI 318-99 and ACI 318R-99, no comparison is made, either in the body of the Standard or its Commentary, of research results for test modules satisfying ACI 318-99 with those for modules that, although not satisfying ACI 318-99, do satisfy the Standard. Such comparisons, both experimental and analytical, are available in the references of the Commentary.

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

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INTRODUCTION

For seismic design, ACI 318-99 specifies in Section 21.2.1.5 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 that satisfies the requirements of this chapter." This Standard defines the minimum experimental evidence that shall be provided in order to validate the use, in regions of high seismic risk or for structures assigned to satisfy high seismic performance or design categories, of a weak beam/strong column moment frame not satisfying the requirements of Chapter 21 of ACI 318-99.

Consistent with the ACI 318-99 requirement for analysis, this Standard specifies that, before the testing mandated by the Standard, a design procedure shall have been developed for

prototype frames having the generic form for which acceptance is sought and that design procedure shall be used to proportion the test modules. Further, the Standard assumes that the prototype frames have forms that are essentially regular, having no significant physical discontinuities in plan or in vertical configuration or in their lateral-force-resisting systems, and that the frames satisfy some, but not all, of the requirements of Chapter 21. Such frames might, for example, involve use of precast elements, precast prestressed elements, post-tensioned reinforcement, or combinations of those elements and reinforcement. Prescriptive requirements for moment frames constructed with such elements are not included in ACI 318-99. Such frames might also, for example, use alternate methods, other than those specified in Chapter 21, for force transfer through beam-column joints.

The provisions of this Standard are intended to supplement the provisions of Chapter 21 of ACI 318-99 and not to supplant them.

1.0—Notation

Only symbols additional to those in ACI 318-99 are defined.

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

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

E_{pr} = probable lateral resistance of test module determined using actual geometric and material properties of test members, an analysis for probable flexural moment strength of beams based on strain compatibility and including strain-hardening effects in the reinforcement, and a strength reduction factor ϕ of 1.0

λ = column overstrength factor used for test module

θ = drift ratio

β = relative energy dissipation ratio

R1.0—Notation

Only symbols used in this Commentary that are additional to those in Appendix E of ACI 318-99 and Standard T1.1-01 are defined in the following:

A_h = area of hysteresis loop

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

f_l = factor on live load defined in R2.6

h = height of column of test module, in. or mm

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

θ_1, θ_2 = drift ratios at peak lateral resistance for positive, negative, loading 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, negative,

lateral resistance for third cycle of loading sequence (Fig. R2.4)

Δ = lateral displacement, in. or mm. See Fig. R2.1

Δ_a = allowable story drift, in. or mm. See Table 1617.3 of IBC 2000

2.0—Definitions

2.1 Drift ratio—Angular rotation under load of the column chord of the test module with respect to the beam chord, where the chords are the straight lines connecting the centroidal axes of the points of contraflexure in the beam and the column, respectively, or the centroidal axis at the point of contraflexure to the centroid of the beam-column joint in the case where a member extends on one side of the joint only.

R2.1 Where a column exists on both sides of the joint, its chord is defined by the line joining the loading (or support) points. The same is true for a beam that exists on both sides of the joint. If a column or beam exist on one side of the joint only, then the chord is defined by the line joining the end loading (or support) point and the joint centerline.

The drift ratio θ concept is illustrated in Fig. R2.1 for an exterior column-beam module. The position of the module at the start of testing, with its self-weight only acting, is indicated by broken lines. The module is pin supported at A and roller supported at D. The self weight is taken by vertical reactions V_{AD} and V_{DD} . That weight, however, also causes a twisting about the centroid B of the joint so that opposing horizontal reactions, H_{AD} and H_{CD} , develop. Under self weight alone, the pin at C must be constrained to lie on the centroidal axis of the column that passes from C through B to A. That chord is the vertical reference line for drift measurements. The setup also constrains the chord joining the centroid of the joint B and the centroid of the section at D to be horizontal.

For acceptance testing, a lateral force H_{CE} is applied to the column through the pin at C and results in the specimen taking up the deformed shape indicated by solid lines. The lateral force causes reactions H_{AL} at A and V_{DL} at D. The column at C displaces laterally by an amount Δ . The chord defining the reference axis for the beam, however, remains horizontal. The drift ratio is the angular rotation of the column chord with respect to the beam chord and for the setup shown equals Δ/h where h is the column height and equal to the distance between the pin at A and that at C.

2.2 Moment frame—Space frame in which members and joints resist forces through flexure, shear and axial force.

2.3 Overstrength factor—Ratio of the sum of the nominal flexural strengths of the columns at their interfaces with the joint to the sum of the nominal flexural moment strengths of the beams at their interfaces with the same joint.

R2.3 The column overstrength factor λ should be selected so that λE_n is greater than the probable lateral resistance E_{pr} . It is to be expected that the maximum lateral resistance of the test module E_{max} should be similar to E_{pr} . In 21.4.2.2 of ACI 318-99 the ratio of the sum of the moments at the faces of the joint, corresponding to the nominal flexural strengths of the columns framing into that joint, to the sum of the moments at the faces of the joint, corresponding to the

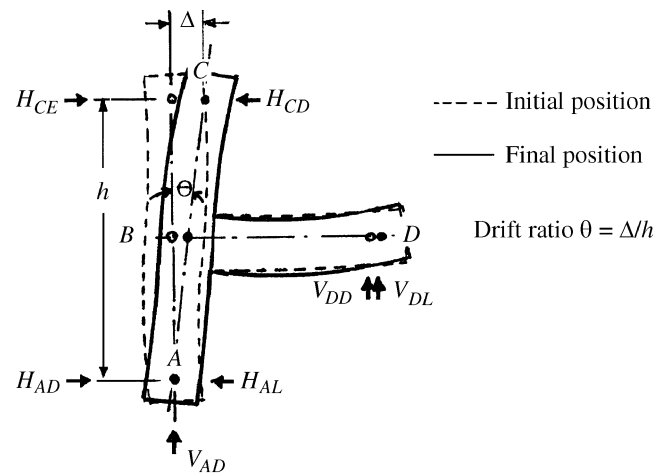


Fig. R2.1—Deformations of exterior column-beam test module.

nominal flexural strengths of the beams framing into that same joint, must exceed 1.2. Further, in T-beam construction, where the slab is under tension under moments at the face of the joint, slab reinforcement within an effective width defined in 8.10 of ACI 318-99 must be assumed to contribute to the flexural strength if the slab reinforcement is developed at the critical section for flexure. Hence the λ specified here is a comparable quantity to, but is not the same quantity as, the 1.2 value specified in ACI 318-99. For application of this Standard, the column overstrength factor is to be specified in the design procedure and, when the contribution of the reinforcement in the slab is considered, the requirement of 21.4.2.2 must be met. There is, however, no requirement to provide a slab on the test module.

Moment strengths should take into account simultaneous application of axial force and direction of loading. The axial forces on the beam and the column should be those causing the largest and the smallest moment strength possible, respectively. Most beams, however, will have zero axial force and for most columns the smallest strength will also be for zero axial force. Directional effects require that the sign of any column axial force and beam bending moments be consistent. For example, for an end joint, column tension effects need only be considered in combination with beam positive moment strength.

Where prestressing steel is used in frame members the stress f_{ps} in the reinforcement at nominal and probable lateral resistance shall be calculated in accordance with 18.7 of ACI 318-99.

2.4 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 stiffness during the first cycle and the peak resistance during the cycle for which the relative energy dissipation ratio is calculated. See 9.1.3.

R2.4 The relative energy dissipation ratio concept is illustrated in Fig. R2.4 for the third cycle to the drift ratio of 0.035. For Fig. R2.4, it is assumed that the test module has

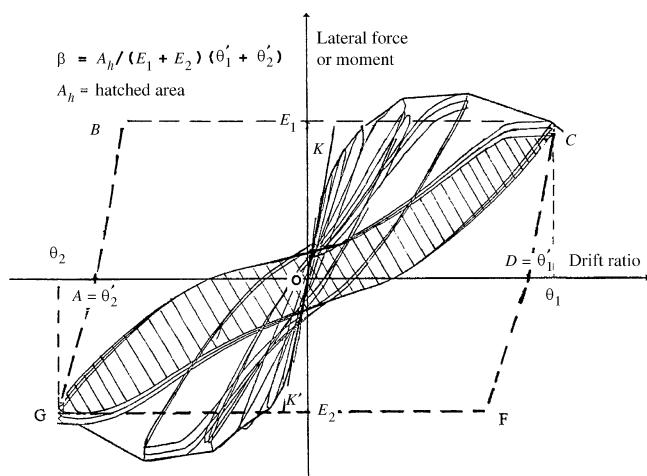


Fig. R2.4—Relative energy dissipation ratio.

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 for the third cycle, A_h , is hatched. 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 ratio concept used in 13.3.3.1 and 13.9.5.2 of the 1997 NEHRP Provisions and Commentary¹ for design and evaluation of seismically isolated structures.

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 drifts ratios θ_1' and θ_2' .

2.5 Test module—Laboratory specimen representing characteristics of typical configuration of intersecting beams and columns of moment frame for which acceptance is sought. See 5.0.

2.6 Toughness—The ability of the entire lateral-force resisting system to maintain structural integrity and continue to carry the required gravity load at the maximum lateral displacements anticipated for the ground motions of a major seismic event.

R2.6 The required gravity load is the value given by the governing building code. Since the purpose of this document is to define acceptance criteria for weak-beam/strong column moment frames not satisfying the requirements of Chapter 21 of ACI 318-99, the response of the beam will generally control the response of the module. In that case, for conformity with both UBC 1997⁷ and IBC 2000¹¹ the required gravity load is $1.2D + f_l L$ where seismic load is additive to gravity forces, and $0.9D$ where seismic load counteracts gravity forces. D is

the effect of dead loads, L is the effect of live loads, and f_l is a factor equal to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 lb/ft^2 (4.79 kN/m^2) where f_l equals 1.0.

3.0—Scope

R3.0—Scope

While only Committee 318 can determine the requirements necessary for frames to meet the provisions of 21.2.1.5 of ACI 318-99, Section 1.4 of ACI 318-99 already permits the building official to accept framing systems other than those explicitly covered by Chapter 21, provided specific tests, load factors, deflection limits, and other pertinent requirements have been established for acceptance of those systems consistent with the intent of the Code. The intent of this document is to provide a framework that establishes the specific tests, etc., appropriate for acceptance, for regions of high seismic risk or for structures assigned to satisfy high seismic performance or design categories of weak beam/strong column moment frames not satisfying all the requirements of Chapter 21. For regions of moderate seismic risk or for structures assigned to satisfy intermediate seismic performance or design categories, less stringent provisions than those specified here are appropriate.

This document assumes that the structural frame to be tested has details differing from those of 21.2 through 21.5 of ACI 318-99 for conventional monolithic reinforced concrete construction. Such frames might, for example, involve the use of precast elements, precast prestressed elements, post-tensioned reinforcement, or combinations of those elements and reinforcement. Alternate methods for force transfer within beam-column joints might also be approved for monolithic or precast moment frame systems based on experimental evidence and analysis using the procedures described in this document.

The fundamental requirement of ACI Code 318-99 for the weak beam/strong column action for moment frames in regions of high seismic risk is retained. The reason is because tests on subassemblages, as envisioned in this document, cannot be extrapolated with confidence to the performance of multistory frames if column sway mechanisms develop in the subassemblage test.

3.1 This document defines minimum acceptance criteria for new reinforced concrete moment frames designed for regions of high seismic risk or for structures assigned to satisfy high seismic performance or design categories, where acceptance is based on experimental evidence and mathematical analysis.

R3.1 This document is not intended for use with existing construction or for use with frames that are designed to conform with all requirements of Chapter 21 of ACI 318-99.¹ These criteria are more stringent than those for frames designed to ACI 318-99, and some frames designed to ACI 318-99 do not meet the 0.035 drift ratio limit.¹²

3.2 Reinforced concrete moment frames, designed on the basis of a weak beam/strong column concept, shall be deemed to have a response that is, as a minimum, at least

equivalent to the response of monolithic frames designed in accordance with 21.2 through 21.5 of ACI 318-99,¹ when both of the following conditions are satisfied:

3.2.1 Tests on frame modules, in accordance with this document, establish the dependable and predictable strength, drift-ratio capacity, relative energy dissipation, and stiffnesses required by the acceptance criteria of 9.0.

R3.2.1 *For acceptance, the results of the tests on each module to be used in the frame must satisfy the criteria of 9.0. In particular, the relative energy dissipation ratio calculated from the measured results for the third cycle between limiting drift ratios of 0.035 must equal or exceed 1/8. Typical relative energy dissipation ratios at 0.030 drift ratios have been reported to be 30, 17, and 10% for reinforced concrete,² hybrid reinforced/prestressed concrete,² and prestressed concrete modules,^{3,4} respectively. In a building frame, as compared to a test module, damping is generally also provided by column hinging at the base of the frame. Further, that hinging is likely to be in a region of monolithic construction or one for which the relative energy dissipation characteristics differ from those of the test module. Hence, the relative energy dissipation ratios for frames with hybrid or prestressed concrete beam sections will probably be greater than the values established from module tests.*

3.2.2 The frame as a whole, based on the results of the tests of 3.2.1 and analysis, shall be demonstrated as able to retain its structural integrity and support its specified gravity loads through peak displacements equal to or exceeding story-drift ratios of 0.035.

R3.2.2 *The criteria of 9.0 are for the test module. In contrast, the toughness criterion of 3.2.2 is for the frame as a whole and can be satisfied only by the philosophy used for the design and analysis of the frame as a whole. The criterion adopted here is similar to that described in R21.2.1 of ACI 318R-99 and the intent is that test results and analyses demonstrate that the structure is still capable of supporting the specified gravity load after cycling through drift ratios of +0.035 to -0.035.*

4.0—Design procedure

R4.0—Design procedure

The test program specified in this document is intended to verify an existing design procedure for a generic type of moment frame system and is not for the purpose of creating basic information on the strength and deformation properties required for design. For a generic system to be accepted based on this document, a rational design procedure must be developed first. That procedure must be based on a rational consideration of material properties and force transfer mechanisms, and its development will probably require preliminary testing that is not part of the validation testing. Because a moment frame is likely to respond inelastically during design-level ground shaking, the design procedure must consider frame configuration, equilibrium of forces, compatibility of deformations, the magnitudes of the lateral drifts, reversed cyclic displacements, and use appropriate constitutive laws for materials that include

considerations of effects of cracking, loading reversals, and inelasticity.

4.1 Before testing, a design procedure shall be developed for prototype moment frames having the generic form for which acceptance is sought. That procedure shall account for effects of material nonlinearity, including cracking, deformations of members and connections, and reversed cyclic loadings.

4.2 The design procedure shall be used to proportion the test modules.

R4.2 *The justification for the small number of test modules is that a rational design procedure is being verified by the test results. Thus, the test modules for the experimental program must be designed using the procedure intended for the prototype moment frame and strengths must be predicted for the test modules before the acceptance testing is started.*

4.3 The overstrength factor used for the columns of the prototype frame shall be not less than that specified in 21.4.2.2 of ACI 318-99.¹

5.0—Test modules

5.1 A minimum of one module shall be tested for each characteristic configuration of intersecting beams and columns in the generic moment frame.

R5.1 *Each characteristic configuration of intersecting beams and columns in the proposed moment frame must be tested. Thus, as a minimum for a one-way, multibay moment frame, modules with the two configurations shown in Fig. R5.1(a) and (b) must be tested. In addition, if the moment frame system includes intersecting one-way frames at corners, then the configuration of Fig. R5.1(c) must also be tested. For two-way frames, testing of additional configurations, representative of interior and exterior two-way intersections, is required. Testing of configurations other than those shown in Fig. R5.1 may be appropriate when it is difficult to realistically model the intended actions using only a half beam or half column. In such cases, a complete bay of the frame should be tested.*

This provision should not be interpreted as implying that only one test will need to be made to qualify a generic system. During the development of that system it is likely that several tests will have been made that have resulted in progressive refinements of the mathematical model used to describe the likely performance of the generic frame and its construction details. Consequently, only one test of each module type, at a specified minimum scale and subjected to specified loading actions, is required to validate the system. Further, if any one of those modules for the generic frame fails to pass the validation testing required by this Standard, then the generic frame has failed the validation testing.

In the generic frame, a slab is usually attached to the beam. However, in conformity with common practice for the sub-assemblages used to develop the provisions of Chapter 21 of ACI 318-99, there is no requirement for a slab to be attached to the beam of the test module. The effect of the presence of the slab should be examined in the development program that precedes the validation testing.

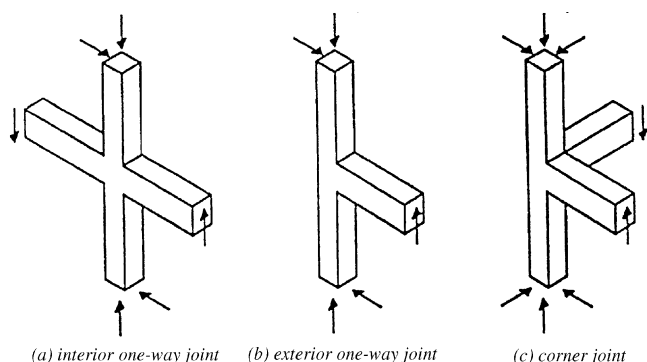


Fig. R5.1—Characteristic intersection configurations and test actions.

5.2 Modules shall have a scale large enough to represent fully the complexities and behavior of the real materials and of the load transfer mechanisms in the prototype frame. Modules shall have a scale not less than one-third full size.

R5.2 Test modules need not be as large as the corresponding modules in the prototype frame. The scale of the test modules, however, must be large enough to capture the full complexities associated with the materials of the prototype frame, its geometry and reinforcing details, and its load transfer mechanisms. For modules involving the use of precast elements, for example, scale effects for load transfer through mechanical connections should be of particular concern.⁵ 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.¹⁰

5.3 The minimum extent of modules on either side of a beam-column joint shall be the distance between the contraflexure points nearest to that joint for both beams and columns for linear elastic lateral load response of the generic moment frame.

R5.3 The points of contraflexure nearest to the joint for the linear elastic lateral loading of frames in high seismic zones are, in general, the midpoints of the members. However, the significance of the magnitude of the gravity load that acts simultaneously with the lateral load may need to be addressed during the validation testing if the development program has demonstrated that effect to be significant.

6.0—Testing agency

Testing shall be carried out by an independent testing agency working under the supervision of a professional engineer experienced in seismic structural design.

R6.0—Testing agency

In accordance with the spirit of the requirements of 1.3.5 and 1.4 of ACI 318-99, it is important that testing be carried out by a recognized independent testing agency and that the testing and reporting be supervised by a professional engineer familiar with the proposed design procedure and experienced in testing and seismic structural design.

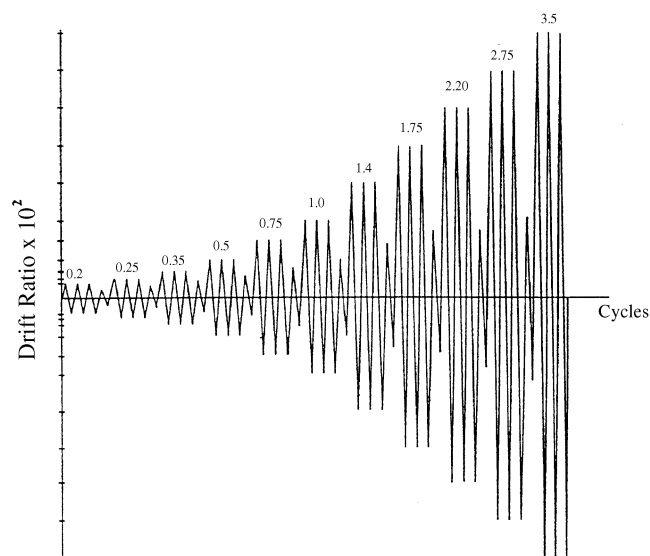


Fig. R7.0—Example of test sequence of displacement controlled cycles.

7.0—Test method

R7.0—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.^{3,4,6}

An example of the test sequence specified in 7.2 through 7.4 is illustrated in Fig. R7.0. The sequence is intended to ensure that displacements are increased gradually in steps that are neither too large nor too small. If 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 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.

The drift capacity of a building frame 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 column simultaneously with the application of the lateral displacements. It is conservative not to apply axial load because, in general, the axial load will be less than the balanced load for frames for which this Standard will be used. The significance of the level of axial loading should be examined during the development phase.

7.1 Test modules shall be subjected to a sequence of displacement-controlled cycles representative of the drifts expected under earthquake motions for that portion of the frame represented by the test module. Cycles shall be to predetermined drift ratios as defined in 7.2, 7.3, and 7.4.

7.2 Three fully reversed cycles shall be applied at each drift ratio.

7.3 The initial drift ratio shall be within the essentially linear elastic response range for the module. Subsequent drift ratios shall be to values not less than one and one-quarter times, and not more than one and one-half times, the previous drift ratio.

7.4 Testing shall continue with gradually increasing drift ratios until the drift ratio equals or exceeds 0.035.

R7.4 *For the response of a structure to the design seismic shear force, current building codes such as UBC-97⁷ and IBC 2000,¹¹ or recommended provisions such as NEHRP-97,¹ specify a maximum allowable drift. Structures designed to meet that drift limit, however, may experience greater drifts during an earthquake equal to the design basis earthquake. 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 2000, 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 factor increases with the expected ductility for the lateral force resisting system of the building. Monolithic moment frames satisfying the requirements of 21.1 through 21.5 of ACI 318-99 are assigned an R value of 8 and an allowable story drift ratio that is dependent on the hazard posed by the building and the building height. 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. When the moment frames are part of a building representing a substantial hazard to human life in the event of a failure, the allowable story drift ratio is 0.020 for frames four stories or less in height and 0.015 for frames greater than four stories in height. If the building failure does not pose a substantial hazard to human life, the corresponding drift ratios are 0.025 and 0.020.

To compensate for the use of the R value, IBC 1617.4.6 requires that the drift determined by an elastic analysis be multiplied by a deflection amplification factor C_d to determine the design story drift and that design story drift must be less than the allowable story drift. For monolithic frames satisfying the requirements of 21.1 through 21.5 of ACI 318-99, C_d is assigned a value of 5.5. Research⁸ has found, however, that design story drift ratios determined in the foregoing manner may be too low. Drift ratios of 8 times IBC-calculated values (rather than 5.5) are more representative of the upper bounds to expected drift ratios. The value of 8 is also in agreement with the finding⁹ that the drift ratio of an inelastic struc-

ture is approximately the same as that of an elastic structure with the same initial period. The value of 8/5.5 times the present IBC limits on calculated drift ratio would lead to a limit on real drift ratios of 0.022 to 0.036. Yet conventional moment frames made from reinforced concrete¹² or steel¹⁰ are unable to achieve the 0.036 limit on a consistent basis. Thus, a value of 0.035, drift ratio A in Fig. R9.1, was chosen as a conservative limit to be satisfied by the test modules.

7.5—Data shall be recorded from the test such that a quantitative, as opposed to qualitative, interpretation can be made of the performance of the module. A continuous record shall be made of test module drift ratio versus column shear force, and photographs shall be taken that show the condition of the test module at the completion of testing for each sequence of three cycles.

R7.5—*In many cases, data additional to the minimum specified in 7.5 may be useful to confirm both design assumptions and satisfactory response. Such data include relative displacements, rotations, curvatures, and strains.*

8.0—Test report

R8.0—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.

8.1—The test report shall contain sufficient evidence for an independent evaluation of the performance of the test module. As a minimum, all of the following information shall be provided:

8.1.1—A description of the theory used to predict test module strength together with predictions of test module nominal lateral resistance E_n and test module probable lateral resistance E_{pr} .

8.1.2—Details of test module design and construction, including engineering drawings.

8.1.3—Specified material properties used for design, and actual material properties obtained by testing.

8.1.4—Description of test setup, including diagrams and photographs.

8.1.5—Description of instrumentation, locations, and purpose.

8.1.6—Description and graphical presentation of applied drift ratio sequence.

8.1.7—Description of observed performance, including photographic documentation, of test module condition at key drift ratios that include the ratios corresponding to first cracking and first crushing of the concrete for both positive and negative loading directions.

8.1.8—Graphical presentation of lateral force versus drift ratio response.

8.1.9—Graphical presentation of relative energy dissipation ratio versus drift ratio.

8.1.10—Test date, report date, name of testing agency, report author(s), supervising professional engineer, and test sponsor.

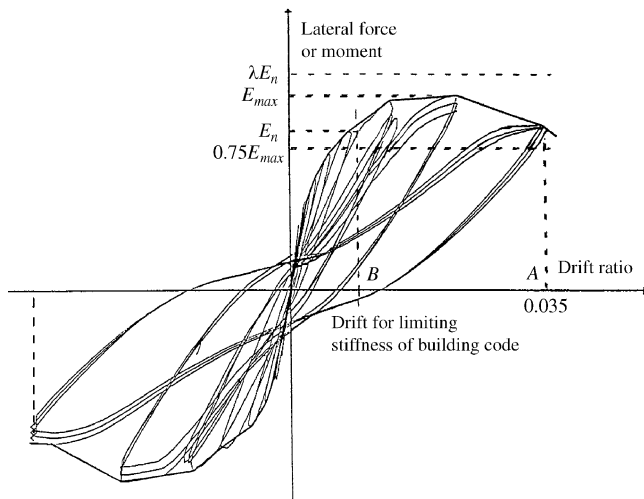


Fig. R9.1—Quantities used in evaluating acceptance criteria.

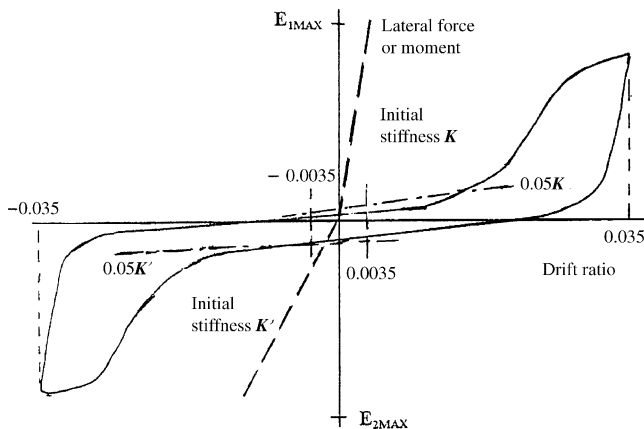


Fig. R9.1.3—Unacceptable hysteretic behavior.

9.0—Acceptance criteria

R9.0—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. R9.1 illustrates the intent of this clause.

9.1 The test module shall be deemed to have performed satisfactorily when all of the following criteria are met for both directions of response:

9.1.1 The test module shall have attained a lateral resistance equal to or greater than E_n before its drift ratio exceeds the value consistent with the allowable story drift limitation of the International Building Code.²

R9.1.1 To provide adequate initial stiffness, Section 9.1.1 specifies that the nominal strength E_n must be developed before the drift ratio exceeds an initial drift ratio consistent with the allowable story drift limitations of IBC 2000. Allowable story drifts Δ_a are specified in Table 1617.3 of IBC 2000 and typical values are reported in R7.4. The limiting initial drift ratio consistent with Δ_a equals $\Delta_a/\phi C_d h$, 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 equal to 0.015, the required deflection amplification factor C_d of 5.5, and ϕ equal to 0.9, the limiting initial drift ratio, B in Fig. R9.1, is 0.003. 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 .

Where nominal strengths for opposite loading directions differ, as is likely for exterior intersections, this criterion applies separately to each direction.

9.1.2 The maximum lateral resistance E_{max} recorded in the test shall have not exceeded λE_n , where λ is the specified overstrength factor for the test column.

R9.1.2 To provide weak beam/strong column behavior, the design procedure must specify an overstrength factor by which the sum of the nominal flexural moment strength of the columns at the faces of the joint exceeds the sum of the nominal flexural moment strengths of the beams at the faces of the same joint and in the same vertical plane. In 21.4.2.2 of ACI 318-99, the somewhat comparable design overstrength factor is required to be equal to or greater than 1.2, but in that case the effect of the contribution of the reinforcement in the slab to the strength of the beams must also be considered. For the generic frame, nominal flexural moment strengths should be calculated according to Chapter 10 and 21.4.2.2 of ACI 318-99. For columns, the nominal flexural strength should be calculated for the factored axial force, consistent with the direction of the lateral force considered, that results in the lowest flexural strength. For proportioning the test modules, the overstrength factors λ calculated on the basis of nominal moment strengths and neglecting column axial load effects should equal or exceed 1.2 when the effect of the contribution of the reinforcement in any slab to the flexural strength of the beam is also considered.

Because of differences between specified and actual yield strengths of reinforcing steel, as well as strain-hardening effects, the design overstrength factor of 1.2 specified in ACI 318-99 may not be sufficient to prevent column yielding in monolithic reinforced concrete construction. For the discretely jointed construction possible with precast elements, strain concentrations and prying actions may cause greater strain hardening effects than for comparable monolithic construction. Further, for hybrid and prestressed frames, where relative energy dissipation ratios lower than those for reinforced concrete frames occur, column yielding is particularly undesirable. Thus, for construction consistent with this document, design overstrength factors greater than 1.2 are desirable. To validate that the columns will not yield the maximum strength developed in the test, E_{max} , must be less than the λE_n .

9.1.3 For cycling at the given drift level at which acceptance is sought, but not less than a drift ratio of 0.035, the characteristics of the third complete cycle shall have satisfied the following:

1. Peak force for a given loading direction shall have been not less than $0.75E_{max}$ for the same loading direction;
2. The relative energy dissipation ratio shall have been not less than 1/8; and

3. The secant stiffness from a drift ratio of -0.0035 to a drift ratio of $+0.0035$ shall have been not less than 0.05 times the stiffness for the initial drift ratio specified in 7.3.

R9.1.3

1. 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 $0.25E_{\max}$ is specified. Where strengths differ for opposite loading directions, this requirement applies independently to each direction.

2. If the relative energy dissipation ratio is less than $1/8$, there may be inadequate damping for the frame as a whole. Oscillations may continue for a considerable time after an earthquake, producing low-cycle fatigue effects, and displacements may become excessive.

3. 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 0.035 that has the form shown in Fig. R9.1 is acceptable. At zero drift ratio, the stiffnesses for positive and negative loading are about 7 and 11%, respectively, of the initial stiffnesses. Those values satisfy 9.1.3.³ An unacceptable hysteresis loop form would be that shown in Fig. R9.1.3 where the stiffness around zero drift ratio is unacceptably small for positive, but not for negative, loading.

10.0—Standard references

1. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (318R-99)," American Concrete Institute, Farmington Hills, Mich., 1999, 391 pp.
2. 2000 International Building Code, Final Draft, July 1998, International Code Council, Falls Church, Va.

R10.0—Commentary references

1. "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1—Provisions, 1997 Edition," Federal Emergency Management Agency, FEMA 302, Washington, D.C., Feb. 1998, 337 pp. and Part 2—Commentary, FEMA 303, Feb. 1998, 362 pp.
2. Cheok, G. S.; Stone, W. C.; and Nakaki, S. D., "Simplified Design Procedure for Hybrid Precast Concrete Connections," NISTIR 5765, NIST, Gaithersburg, Md., Feb. 1996, 81 pp.
3. Stanton, J. F., and Mole, A., "A Hybrid Precast Prestressed Concrete Frame System," Fourth Meeting of U.S.-Japan Joint Technical Coordinating Committee on PRESSS, Tsukuba, Japan, May 1994, 24 pp.
4. Priestley, M. J. N., and Tao, J. R., "Seismic Response of Precast Prestressed Concrete Frames with Partially Debonded Tendons," *PCI Journal*, V. 38, No. 1, Jan.-Feb. 1993, pp. 58-69.
5. French, C. W.; Hafner, M.; and Jayashanker, V., "Connections between Precast Elements—Failure within Connection Region," *Journal of Structural Engineering*, ASCE, V. 115, No. 12, Dec. 1989, pp. 3171-3192.
6. Priestley, M. J. N., "The PRESSS Program—Current Status and Proposed Plans for Phase III," *PCI Journal*, V. 41, No. 2, Mar.-Apr. 1997, pp. 22-33.
7. International Conference of Building Officials, "Uniform Building Code: V. 2, Structural Engineering Design Provisions," Whittier, Calif., May 1997.
8. Uang, C.-M., and Maarouf, A., "Seismic Displacement Amplification Factor in Uniform Building Code," SEAONC Research Bulletin Board, BB93-3, June 1993, pp. B1-B2, and "Displacement Amplification Factor for Seismic Design Provisions," *Proceedings of Structures Congress*, ASCE, V. 1, Irvine, Calif., 1993, pp. 211-216.
9. Veletsos, A. S., and Newmark, N. M., "Effects of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions," *Proceedings*, V. 2, 2WCEE, Tokyo, Japan, 1960, pp. 895-912.
10. Engelhardt, M. D., and Sabol, T. A., "Testing of Welded Steel Moment Connections in Response to the Northridge Earthquake," *Progress Report to the AISC Advisory Subcommittee on Special Moment Resisting Steel Frame Research*, Oct. 1994.
11. 2000 International Building Code, Final Draft, July 1998, International Code Council, Falls Church, Va., 22041-3401.
12. Cheok, G. S.; Stone, W. C.; and Kunnath, S. K., "Seismic Response of Precast Concrete Frames with Hybrid Connections," *ACI Structural Journal*, V. 95, No. 5, Sept.-Oct. 1998, pp. 527-539.