APPENDIX A—DESIGN VERIFICATION USING NONLINEAR RESPONSE HISTORY ANALYSIS

CODE

A.1—Notation and terminology

A.1.1 *Notation*

B = bias factor to adjust nominal strength to seismic target reliabilities

D_u = ultimate deformation capacity; the largest deformation at which the hysteresis model is deemed valid given available laboratory data or other substantiating evidence

$\sqrt{f'_{ce}}$ = square root of expected compressive strength of concrete, psi

f_{ce}' = expected compressive strength of concrete, psi

f_{ue} = expected tensile strength for nonprestressed reinforcement, psi

f_{ye} = expected yield strength for nonprestressed reinforcement, psi

ℓ_p = plastic-hinge length for analysis purposes, in.

R_{ne} = expected yield strength

V_{ne} = expected shear strength, lb

θ_y = yield rotation, radians

ϕ = seismic resistance factor for force-controlled actions

A.1.2 *Terminology*

distributed plasticity (fiber) model—component model consisting of discrete fibers explicitly representing nonlinear stress-strain or force-deformation responses.

structural wall panel zone—portion of a structural wall common to intersecting wall segments where forces from adjacent wall segments are resolved.

The following actions shall be as defined by ASCE/SEI 7 Chapter 16:

action, deformation-controlled
action, force-controlled
action, force-controlled critical
action, force-controlled ordinary
action, force-controlled noncritical

A.2—Scope

A.2.1 This appendix shall supplement the requirements of Chapter 16 of ASCE/SEI 7 when performing nonlinear response history analysis to determine the design of earthquake-resistant concrete structures.

A.2.2 The provisions of Appendix A shall be in addition to the provisions of Chapters 1 through 26.

A.2.3 This appendix shall be used in conjunction with Chapter 16 of ASCE/SEI 7 for additional general requirements, ground motions, load combinations, modeling, and analysis for design of new reinforced concrete structures, including:

(a) Structural systems designated as part of the seismic force-resisting system, including diaphragms, moment-resisting frames, structural walls, and foundations.

COMMENTARY

RA.1—Notation and terminology

RA.1.2 *Terminology*

Force-controlled and deformation-controlled actions are classified in A.7 for design using nonlinear analysis of concrete structures.

RA.2—Scope

RA.2.3 This appendix is intended to complement documents such as Chapter 16 of ASCE/SEI 7, TBI (2017), and LATBSDC (2023). This appendix provides requirements specific to nonlinear response history analysis and design of concrete members. For additional analysis and modeling requirements that are not specific to concrete members, refer to Chapter 16 of ASCE/SEI 7, TBI (2017), and LATBSDC (2023).

A Nonlinear

CODE

(b) Members not designated as part of the seismic force-resisting system but required to support other loads while sustaining deformations and forces associated with earthquake effects.

A.2.4 All concrete structures designed or verified by this Appendix shall be proportioned and detailed as required by Chapter 18 and the requirements of A.12 when applicable.

A.2.5 It shall be permitted to use the provisions of Appendix A to demonstrate the adequacy of a structural system as required by 18.2.1.7.

A.2.6 Independent structural design review consistent with A.13 shall be required for use of Appendix A.

A.2.7 The licensed design professional shall provide justification for any interpretation required for the application of Appendix A, and if accepted by the independent structural design reviewers, justification shall be provided to the building official for acceptance.

A.3—General

A.3.1 Action Classification and Criticality in A.7, and Acceptance Criteria in A.10 and A.11 provide a comprehensive design approach following the intent of Chapter 16 of ASCE/SEI 7 and the general building code, and shall take precedence over those of Chapter 16 of ASCE/SEI 7.

A.4—Earthquake ground motions

A.4.1 Nonlinear response history analysis shall include the effects of horizontal earthquake ground motions.

A.4.2 Vertical earthquake ground motion shall be considered simultaneously with horizontal earthquake ground motions where inclusion of vertical ground motion will substantially affect the structural design requirements.

A.4.3 Earthquake ground motion acceleration histories shall be selected and modified in accordance with procedures established by the general building code.

COMMENTARY

RA.2.7 It is anticipated that the initial design of a earthquake-resistant structure will be performed using elastic analysis combined with engineering judgment. A nonlinear response history analysis following the requirements of this Appendix can then be performed to demonstrate the design, which may not fully comply with all provisions of ASCE/SEI 7 or the general building code.

R.3—General

RA.3.1 Due to inconsistencies between ACI CODE-318 and Chapter 16 of ASCE/SEI 7 in the approach to Action Classification and Acceptance Criteria for concrete members, the requirements in this Appendix take precedence over those of ASCE/SEI 7.

RA.4—Earthquake ground motions

RA.4.1 Nonlinear response history analysis commonly is performed using two horizontal components of earthquake ground motion applied to a three-dimensional model of the building.

RA.4.2 Structures with vertical discontinuities in the gravity-load-resisting systems can experience vertical earthquake response that can affect building performance. Examples include columns or walls that terminate on beams or slabs. Some structures with long spans or long cantilevers can be sensitive to vertical ground motion. Engineering judgment should be exercised when considering the sensitivity of structures to vertical ground motions.

RA.4.3 The analysis procedures in Appendix A are based on ground motion selection and scaling consistent with Chapter 16 of ASCE/SEI 7, which includes scaling to a risk-targeted maximum considered earthquake ground acceleration. ASCE/SEI 7 describes appropriate procedures for selection and modification of earthquake ground motions in terms of acceptable hazard and risk levels.

CODE

A.5—Load factors and combinations

A.5.1 Load combinations for nonlinear response history analysis shall conform to the requirements of the general building code.

COMMENTARY

RA.5—Load factors and combinations

RA.5.1 Load combinations for response history analysis used in conjunction with this Appendix are intended to be similar to those of Chapter 16 of ASCE/SEI 7, TBI (2017), and LATBSDC (2023).

For nonlinear response history analysis, the principles of linear superposition do not apply. Therefore, it would be incorrect to conduct separate analyses considering various loads and then combine the load effects. Instead, it is necessary to conduct an analysis for each factored load combination and take the design value as the envelope of the analysis results. For any nonlinear analysis including earthquake effects, gravity loads are to be applied to the model first and then the ground shaking simulations are applied in the presence of the gravity loads.

There is a low probability that maximum considered earthquake shaking and factored design gravity load combinations of the general building code will occur simultaneously. A more representative load combination is the occurrence of expected, realistic gravity loading combined with maximum considered earthquake shaking.

One load combination is typically considered for analysis, which includes expected dead load concurrent with expected live load and Maximum Considered Earthquake shaking. Chapter 16 of ASCE/SEI 7 requires consideration of a second load combination without live load. It should be noted that this case will seldom govern the design of a tall building.

Accidental torsion is not commonly considered in cases where linear analysis indicates that torsional irregularities are negligible.

Load combinations used in the nonlinear analysis may differ from load combinations used to evaluate force-controlled actions (refer to A.11).

A.6—Modeling and analysis

A.6.1 Models for analysis shall be three-dimensional and shall conform to the requirements of the general building code.

A.6.2 Modeling of member nonlinear behavior, including effective stiffness, expected strength, expected deformation capacity, and hysteresis under force or deformation reversals, shall be substantiated by applicable physical test data and shall not be extrapolated beyond the limits of testing.

A.6.3 Degradation in member strength or stiffness shall be included in the numerical models unless it can be demonstrated that the demand is not sufficiently large to produce these effects. If degradation in component strength is included in the numerical model, the model formulation shall be such that structural deformation at onset of strength loss is not affected by mesh configuration in the finite element model.

RA.6—Modeling and analysis

RA.6.2 Multiple element formulations and material models are appropriate for use in inelastic dynamic analysis of concrete structures. ASCE/SEI 41, ACI PRC-374.3, ACI CODE-369.1, and NIST GCR 17-917-46 provide guidance on modeling and defining model parameters. Selecting model parameters at the mean value of experimental data, as is recommended by the aforementioned documents, avoids skewing analysis results and produces a more reliable evaluation of concrete building response.

RA.6.3 The model mesh size selected should allow determination of the structural responses in sufficient detail and with sufficient accuracy. Some systems will exhibit mesh-dependent response, with a reduction in mesh size resulting in reduced deformation capacity and more rapid strength loss. For these systems, material softening should be defined using a measure of mesh size, or the chosen material model

A Nonlinear

CODE

COMMENTARY

parameters and mesh size should be shown, using an appropriate experimental data set, to provide accurate simulation of onset of strength loss.

A.6.4 For structural walls with aspect ratio $h_w/\ell_w \geq 2$, the numerical model of the wall and its connection to surrounding elements shall represent kinematic effects associated with wall rotation and uplift, including the effect of migration of the neutral axis as a function of applied axial force and lateral deformation, unless it can be demonstrated that such effects do not affect the structural design requirements.

A.7—Action classification and criticality

RA.7—Action classification and criticality

A.7.1 All actions shall be classified as deformation-controlled or force-controlled in accordance with A.7.2 and A.7.3.

A.7.2 *Deformation-controlled actions*

RA.7.2 *Deformation-controlled actions*

A.7.2.1 Deformation-controlled actions shall satisfy the requirements of A.10.

A.7.2.2 The following shall be designated as deformation-controlled actions:

(a) Moment in beams, coupling beams, and slab-column connections
(b) Shear in diagonally reinforced coupling beams that meet the requirements of 18.10.7.4
(c) Moment in columns when combined with axial force for columns meeting the requirements of 18.7.4, 18.7.5, and 18.7.6
(d) Moment in walls when combined with axial force for walls controlled by tensile yielding of longitudinal reinforcement
(e) Axial tensile force
(f) Other actions accepted by the independent structural design reviewer(s) based on substantiating test data or analysis

RA.7.2.2 Similar to the requirements of 18.14.3.3, if columns are detailed with sufficient confinement and reinforcement detailing, column moment can be evaluated as a deformation-controlled action rather than as a force-controlled action.

A.7.3 *Force-controlled actions*

RA.7.3 *Force-controlled actions*

A.7.3.1 Force-controlled actions shall satisfy the requirements of A.11.

A.7.3.2 The following shall be designated as ordinary force-controlled actions:

(a) Shear and moment in perimeter basement walls
(b) In-plane shear in non-transfer diaphragms
(c) In-plane normal forces in diaphragms other than collectors
(d) Moment in shallow foundation members, including spread footings and mat foundations
(e) Moment in deep foundation members

RA.7.3.2 For diaphragm shear to be considered an ordinary force-controlled action, the shear should not be related to a transfer of force between lateral-force-resisting system components.

CODE

A.7.3.3 Noncritical force-controlled actions shall be designated as actions in any component where failure will not result in: (a) collapse of the structure; (b) loss of the earthquake resistance of the structure; and (c) falling hazard.

A.7.3.4 All actions not designated as deformation controlled, ordinary force-controlled, or noncritical force-controlled shall be classified as critical-force controlled.

A.8—Effective stiffness

A.8.1 Member stiffness shall include effects of deformations due to flexure, shear, axial elongation or shortening, and reinforcement slip along its development length.

A.8.2 If cracking is anticipated as a result of combined effects of applied forces, displacements, and volume change effects associated with shrinkage, temperature, or creep, effects of concrete cracking on effective member stiffness shall be modeled.

A.8.3 If yielding of reinforcement or nonlinear response of concrete is anticipated as a result of combined effects of applied forces, displacements, and volume change effects associated with shrinkage, temperature, or creep, the structural model shall be capable of representing member stiffness for loading near the onset of inelastic response, as well as behavior past the onset of inelastic response.

COMMENTARY

RA.8—Effective stiffness

RA.8.1 Software for nonlinear analysis generally is capable of directly modeling deformations due to flexure, shear, and axial elongation or shortening. Additional deformation may occur due to slip of longitudinal reinforcement from adjacent anchorages. Such effects commonly occur where beams frame into beam-column joints or walls, where columns frame into beam-column joints or foundations, and where walls frame into foundations. If such effects are considered important to the performance of the structure, appropriate assumptions should be included in the analytical model, either directly or by adjustment of flexural stiffness.

RA.8.2 Effects of cracking on stiffness reduction can be considered directly by using models that represent stiffness reduction as calculated stress reaches the cracking stress or indirectly by reducing the effective stiffness relative to the gross-section stiffness. Where the latter approach is used, the degree of stiffness reduction should be consistent with the degree of cracking anticipated under earthquake loading. Structural walls that are lightly cracked, including basement walls, have traditionally been modeled using effective flexural stiffness in the range 0.5 to 1.0 times gross-section stiffness. Diaphragms at major force transfer levels are commonly modeled using effective axial stiffness in the range 0.10 to 0.5 times gross-section stiffness. TBI (2017) and LATBSDC (2023) provide additional effective stiffness recommendations while NIST GCR 17-917-46v1 (NIST 2017a) and NIST GCR 17-917-46v3 (NIST 2017b) provide more detailed guidance on modeling of diaphragms and frame elements.

For stiffness of beams, columns, and structural walls other than basement walls, refer to RA.8.3.

RA.8.3 If calculations indicate nonlinear response under load combinations including earthquake effects, the nonlinear model should be capable of representing an effective secant stiffness from zero loading to a point corresponding to yield-level forces (slope from A to B in Fig. RA.8.3). The model should also be capable of representing stiffness reduction past the yield point. Degradation in element strength or stiffness should be included in the analytical model unless it can be demonstrated that the demand is not sufficiently large as to produce these effects.

A Nonlinear

CODE

A.8.4 It shall be permitted to represent member stiffness near the onset of inelastic response using an effective stiffness based on analysis substantiated by physical test data. Alternatively, it shall be permitted to represent member stiffness near the onset of inelastic response using the effective stiffness values in Table A.8.4.

COMMENTARY

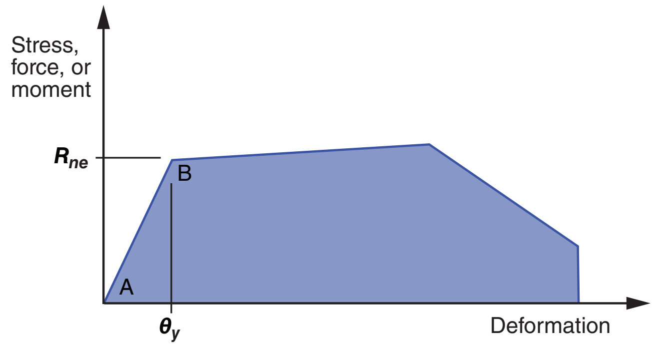

Fig. RA.8.3—Generalized force-deformation relations.

RA.8.4 The effective stiffness values are intended to represent the slope from A to B in Fig. RA.8.3, where B corresponds to expected yield strength. Effective stiffness values for beams and columns are based on Elwood et al. (2007), and incorporate the effects of reinforcement slip along the development length. Tabulated values for structural walls are appropriate to use where the wall is represented by a line element. In some building models, structural walls will be represented by distributed fiber models, in which case the fiber model should directly represent effects of concrete cracking and reinforcement yielding, such that the stiffness values in Table A.8.4 do not apply. Basement walls are unlikely to respond at yield-level forces; therefore, larger stiffness values may be more applicable than those in Table A.8.4 for walls. Diaphragm stiffnesses provided in Table A.8.4 represent typical values. Prestressed and nonprestressed diaphragms mainly resisting single-floor in-plane earthquake forces are commonly modeled as rigid, as allowed by ASCE/SEI 7. Diaphragms transferring relatively large in-plane earthquake forces from multiple floor levels can have effective stiffnesses somewhat lower than those represented in Table A.8.4. In cases where analysis results are sensitive to diaphragm stiffness assumptions, it may be prudent to "bound" the solution by analyzing the structure using a range of diaphragm stiffnesses and selecting the design values as the larger forces from the two analyses. Coupling beam effective stiffnesses are intended to represent values for beams cast monolithically with floor slabs. Values are based on equations presented by Vu et al. (2014), but are adjusted to account for the presence of a slab, differences in modeling approach, and typical shear levels (TBI 2017). Engineering judgment should be used to evaluate effective shear stiffness values, noting that due to typical software implementation limitations, gross area is used in lieu of effective area.

CODE

COMMENTARY

Table A.8.4—Effective stiffness values[1]

Component		Axial	Flexural	Shear
Beams	nonprestressed	$1.0E_cA_g$	$0.3E_cI_g$	$0.4E_cA_{shear}$
	prestressed	$1.0E_cA_g$	$1.0E_cI_g$	$0.4E_cA_{shear}$
Columns with compression caused by design gravity loads[2]	$\geq 0.5A_gf_c'$	$1.0E_cA_g$	$0.7E_cI_g$	$0.4E_cA_{shear}$
	$\leq 0.1A_gf_c'$ with tension	$1.0E_cA_g$ (compression) $1.0E_sA_{st}$ (tension)	$0.3E_cI_g$	$0.4E_cA_{shear}$
Structural walls[3]	in-plane	$1.0E_cA_g$	$0.35E_cI_g$	$0.2E_cA_{shear}$
	out-of-plane	$1.0E_cA_g$	$0.25E_cI_g$	$0.4E_cA_{shear}$
Diaphragms (in-plane only)[4]	nonprestressed	$0.25E_cA_g$	$0.25E_cI_g$	$0.25E_cA_{shear}$
	prestressed	$0.5E_cA_g$	$0.5E_cI_g$	$0.4E_cA_{shear}$
Coupling beams	with or without diagonal reinforcement	$1.0E_cA_g$	$0.07\left(\frac{\ell_n}{h}\right)E_cI_g \leq 0.3E_cI_g$	$0.4E_cA_{shear}$
Mat foundations	in-plane	$0.5E_cA_g$	$0.5E_cI_g$	$0.4E_cA_{shear}$
	out-of-plane[5]		$0.5E_cI_g$	

[1]Tabulated values for axial, flexural, and shear shall be applied jointly in defining effective stiffness of an element, unless alternative combinations are justified.
[2]For columns with axial compression falling between the limits provided, flexural stiffness shall be determined by linear interpolation.
[3]Tabulated values are appropriate where members are modeled using line elements to represent their properties.
[4]Diaphragms shall be permitted to be modeled as rigid in-plane if this does not result in differences in analysis outcomes.
[5]Specified stiffness values for mat foundations pertain for the general condition of the mat. Where the wall or other vertical members imposed sufficiently large forces, including local force reversals across stacked wall openings, the stiffness values may need to be reduced.

A.8.5 In beam-column joints if joint flexibility is not modeled explicitly, it shall be permitted to model joint flexibility implicitly by defining the effective stiffness of beams and columns framing into the joint to include joint flexibility and by introducing beam and column rigid end offsets that extend to the center of the joint.

RA.8.5 In reinforced concrete frames detailed to resist earthquake forces, joints are not expected to experience significant degradation. In lieu of a more rigorous representation of joint shear stiffness, rigid offsets of beam and column members extending the length of the joint dimensions are permitted (Birely et al. 2012). A sensitivity study on stiffness assumptions indicates that overall building stiffness may be more sensitive to the choice of effective stiffness for frame and wall members than for joints (Kwon and Ghannoum 2016). The rigid joint offset approach is compatible with the effective stiffness values presented in Table A.8.4, which account for the softening effects of longitudinal bar slip within the joints.

A.8.6 If beams other than coupling beams are cast monolithically with slabs, the effective slab width defined in 6.3.2 shall be included in the evaluation of beam flexural and axial stiffnesses.

A.9—Expected material strength

A.9.1 Expected material strength shall be defined based on applicable project-specific data or data from projects using similar materials and construction. If applicable data are not available, the expected material strengths in Table A.9.1 shall be permitted.

RA.9—Expected material strength

RA.9.1 The multiplier on f_c' may be smaller for high-strength concrete when higher quality control measures are in place or when fly ash or other additives are used. Refer to ACI PRC-232.2 for discussion of impacts of fly ash. Default values for other steel grades have not been provided in Table A.9.1 due to insufficient data.

A Nonlinear

CODE

Table A.9.1—Expected material strengths

Material		Expected strength	
Concrete		$f_{ce}' = 1.3f_c'$[1]	
Reinforcing steel		Expected yield strength, f_{ye}, psi	Expected tensile strength, f_{ue}, psi
A615	Grade 60	70,000	106,000
A706	Grade 60	69,000	95,000
	Grade 80	85,000	112,000

[1]Expected strength f_{ce}' is strength expected at approximately 1 year or longer.

A.10—Acceptance criteria for deformation-controlled actions

A.10.1 Deformations in any of the response history analyses shall not exceed the ultimate deformation capacity D_u unless (a) or (b) is satisfied.

(a) The analysis assumes the strength associated with this mode of deformation is negligible for the remainder of that analysis, and the structure is evaluated for stability and strength.
(b) The analysis is considered to have an unacceptable response as defined by ASCE/SEI 7.

A.10.2 D_u shall be determined by (a), (b), or (c):

(a) D_u of the component shall be taken as the valid range of modeling as demonstrated by comparison of the hysteresis model with suitable laboratory test data including the appropriate gravity load effect.
(b) If special structural walls are modeled using distributed plasticity (fiber) models, D_u shall be evaluated using the average vertical strain. The strain shall be evaluated over a height of the plastic hinge length, ℓ_p , where ℓ_p is the longer of (i) and (ii):

(i) $\ell_p = 0.2\ell_w + 0.03h_w$ (A.10.2.a)

(ii) $\ell_p = 0.08h_w + 0.00015f_yd_b$ (A.10.2.b)

but not exceeding the story height, where d_b and f_y are determined based on the wall longitudinal reinforcement.

(c) If structural components are modeled using lumped plasticity (concentrated hinge) or distributed plasticity (fiber) models, D_u shall be permitted to be in accordance with ACI CODE-369.1 or as substantiated by laboratory testing.

COMMENTARY

RA.10—Acceptance criteria for deformation-controlled actions

RA.10.1 These acceptance criteria are consistent with the component acceptance criteria in TBI (2017), which are different from those in ASCE/SEI 7 and LATBSDC (2023). More detailed discussion regarding the differences of evaluation approaches of deformation-controlled actions in ASCE/SEI 7 and TBI (2017) are provided in TBI (2017).

RA.10.2 Ultimate deformation capacity D_u is typically obtained from statistical analysis of the available test data and can be closely related to Collapse Prevention Acceptance Criteria in ACI CODE-369.1 and ASCE/SEI 41. $D_u$is based on the deformation where substantial loss of gravity load-carrying capacity occurs or, if tests do not progress to this deformation, the limiting deformation for which testing was performed. An example of D_u in the hysteresis curve of an analysis model is shown in Fig. RA.10.2. The Collapse Prevention Acceptance Criteria in ACI CODE-369.1 and ASCE/SEI 41 are typically less than mean experimental values due to scatter in data sets used to develop these criteria. The ASCE/SEI 41 approach also evaluates deformation as the mean of the maximum absolute response from each response history analysis. Appendix A, however, evaluates deformation as the maximum of any of the response history analyses.

Hysteresis behavior of the structural component simulated using fiber material models should be evaluated and adjusted using experimental data for the range of deformation demands and behaviors simulated in the analyses. ACI PRC-374.3 and ACI CODE-369.1 provide nonlinear modeling parameters that can be the basis for hysteresis shape based on experimental data. Figure RA.10.2 shows a hysteresis curve generated using adjusted fiber material models based on such nonlinear modeling parameters to simulate the component behavior observed in laboratory tests.

If D_u is defined by average strain, the length over which strain is defined in the analysis should be consistent with the length over which strain limits are established from experi-

CODE

A.11—Expected strength for force-controlled actions

A.11.1 Force-controlled actions shall be evaluated in accordance with the general building code, with expected strength taken as $\phi_s BR_n$.

COMMENTARY

mental data or are specified in documents such as ASCE/SEI 41, ACI CODE-369.1, TBI, or LATBSDC.

Sufficient number of fibers along the cross section should be used to allow the strain values at fiber centerlines to be extrapolated to locations where strain values are calculated to compare with strain limits, such as, at the extreme edge of the wall compression zone.

For structural walls or coupling beams modeled using fiber elements, deformation acceptance criteria can be represented in either a strain or member deformation basis. The strain results can be obtained directly from the fiber model. The member deformation results, such as plastic hinge rotation, story drift, or chord rotation, can be obtained by aggregated deformation over a group of fiber elements representing the member. Plastic hinge length Eq. (A.10.2a) and (A.10.2b) for walls are from Paulay and Priestley (1992).

An example of acceptance criteria for strain limits is provided in TBI (2017). The unconfined concrete model includes a peak stress at a compressive strain of 0.002, with a descending backbone to 50% of the peak stress value at a compressive strain of 0.003 (the ultimate deformation capacity D_u). The confined concrete model, used where confinement meeting the requirements of 18.10.6.4(e) and (f) are provided, includes a peak stress at a compressive strain 0.008, with a descending backbone to 80% of the peak stress value at a compressive strain of 0.015 (the ultimate deformation capacity D_u). The longitudinal reinforcement tensile strain limit of 0.05 (the ultimate deformation capacity D_u) is based on tensile rupture with consideration of low-cycle fatigue effects, which is corroborated by Segura and Wallace (2018).

Additional references for ultimate deformation capacity, such as ACI CODE-369.1, TBI (2017), and LATBSDC (2023), may be used subject to approval of the independent structural design review.

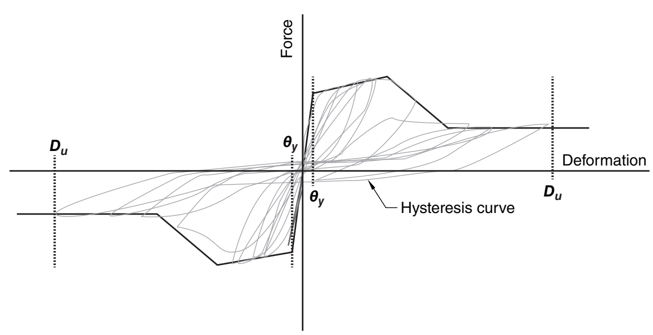

Fig. RA.10.2—D_u *in response hysteresis from an analysis model.*

RA.11—Expected strength for force-controlled actions

RA.11.1 Currently, strength reduction factors ϕ are not specifically calibrated to the seismic reliability targets specified in ASCE/SEI 7. Rather, these strength reduction factors are calibrated to the target reliabilities for other loads

CODE

COMMENTARY

(ASCE/SEI 7-22 Table 1.3-1). The bias factor B is provided to adjust the resistance factors specified by the materials standards to the seismic target reliabilities, considering the inherent bias in the nominal strength equations contained in the materials standards. This bias is a function of both the ratio of expected material strength to minimum specified strength and also inherent conservatism in the predictive equations specified by the materials standards.

A.11.2 ϕ_s shall be in accordance with Table A.11.2, with ϕ determined in accordance with Chapter 21, except that 21.2.4.1 shall not apply.

Table A.11.2—Seismic resistance factor

Force-controlled action	ϕ_s
Critical	ϕ
Ordinary	$\phi/0.9 \leq 1.0$
Noncritical	$\phi/0.85 \leq 1.0$

RA.11.2 For ordinary and noncritical actions, the resistance factors are relaxed in order to accept a higher probability of failure.

More detailed discussion regarding the differences of evaluation approaches of force-controlled actions in ASCE/SEI 7, TBI (2017), and LATBSDC (2023) are provided in TBI (2017) and LATBSDC (2023). Additional background on this approach is provided in Wallace et al. (2013) and Kim and Wallace (2017).

A.11.3 A.11.3 Bias factor, B, shall be taken as 1.0. Alternatively, it shall be permitted to calculate B using Eq. (A.11.3):

$$B = 0.9R_{ne}/R_n \geq 1.0 \quad \text{(A.11.3)}$$

A.11.3.1 Nominal strength, R_n, shall be in accordance with Chapter 18, 22, or 23.

A.11.3.2 The expected strength, R_{ne}, is permitted to be defined in accordance with the nominal strength provisions of Chapters 18, 22, or 23, with f_{ce}' substituted for f_c' and f_{ye} substituted for f_y or f_{yt}, except as provided in A.11.3.2.1 and A.11.3.2.2.

A.11.3.2.1 For structural walls where $h_w/\ell_w \geq 2$ meeting (a) through (d), the requirements of A.11.3.2.1.1 and A.11.3.2.1.2 shall apply.

(a) Wall is modeled with fiber elements in accordance with A.10.2(b)
(b) Strains calculated as the mean of the maximum demand from a suite of response history analyses
(c) Calculated concrete compressive strain < 0.005
(d) Calculated longitudinal tensile strain < 0.01

RA.11.3.2.1 The shear strength determined from these provisions is applicable only to walls with relatively low flexural ductility demands (Wallace et al. 2013; LATBSDC 2023).

A.11.3.2.1.1 $V_{ne} = 1.5A_{cv}(2\lambda\sqrt{f_{ce}'} + \rho_t f_{ye})$

A.11.3.2.1.2 For all vertical wall segments sharing a common lateral force, V_{ne} shall not be taken greater than $12A_{cv}$. For any individual vertical wall segments, V_{ne} shall not be taken greater than $15A_{cv}\sqrt{f_c'}$.

A.11.3.2.2 For structural wall panel zones, V_{ne} shall be calculated in accordance with A.11.3.2.1.1. V_{ne} shall not be taken greater than $25A_{cv}\sqrt{f_c'}$.

CODE

A.12—Enhanced detailing requirements

A.12.1 If the mean maximum deformation from the set of response history analyses exceeds $0.5D_u$ of confined concrete, members shall be subject to the added detailing requirements of this section.

A.12.2 *Special moment frames*

A.12.2.1 For beams of special moment frames, the spacing of transversely supported flexural reinforcing bars as required by 18.6.4.2 shall not exceed 8 in.

A.12.2.2 The sum of the column strengths at any joint as required by 18.7.3.2 shall be at least 1.4 times the sum of the beam strengths at the joint.

A.12.2.3 For tied columns of special moment frames, every longitudinal bar shall have lateral support by a corner of a hoop or a seismic hook as required in 18.7.5.2(f) regardless of axial load or concrete strength.

A.12.2.4 When deformations of beams of special moment frames exceed $0.5D_u$, the column dimension parallel to the beam longitudinal reinforcement required in 18.8.2.3 shall be increased by 20 percent.

A.12.3 *Special structural walls*

A.12.3.1 Boundary elements shall be provided in accordance with 18.10.6 with transverse reinforcement conforming with A.12.2.3.

A.12.3.2 If boundary elements are required, splices of shear reinforcement shall be made with mechanical or welded splices, or lap splices enclosed in transverse reinforcement spaced at the smaller of $6d_b$ of the spliced bars or 6 in.

A.12.3.3 If the floor or roof slab is shown by analysis to undergo inelastic response at a slab-wall connection, the slab

COMMENTARY

RA.12—Enhanced detailing requirements

RA.12.1 The requirements for earthquake-resisting systems and detailing have been developed over many years using actual earthquake damage observations, research, and engineering judgment. These requirements are codified in ASCE/SEI 7, IBC, and ACI CODE-318. In recent years, enhanced computational abilities allow engineers to model and calculate seismic response in great detail.

Designs that exceed the prescriptive limits of the general building code are sometimes prepared, verified, and justified. In some instances, these new designs have not been tested in strong ground shaking, and there is some concern that these designs may be extrapolating beyond the collective knowledge. Therefore, these enhanced details are provided to improve inelastic response ductility and are appropriate when using Appendix A for designs beyond prescriptive code limits.

RA.12.2 *Special moment frames*

RA.12.2.3 This code has allowed crossties in compression members with a seismic hook at only one end and with crossties alternated recognizing their ease in construction. However, recent earthquakes and research tests have shown that 90-degree hooks do not always provide adequate support (Moehle and Cavanagh 1985).

RA.12.3 *Special structural walls*

CODE

flexural reinforcement shall be extended through the slab-wall joint and anchored for structural integrity.

A.12.3.4 If shear force exceeds $4A_{cv}\lambda\sqrt{f_c'}$, enhanced construction joint detailing shall be provided with thorough roughening of concrete, intermittent shear keys in the concrete, or both, to reduce the possibility of slip along the construction joint.

A.13—Independent structural design review

A.13.1 The analysis and design shall be reviewed by an independent structural design reviewer. The independent structural design reviewer shall act under the direction of the building official.

A.13.2 The independent structural design review shall be performed by one or more individuals acceptable to the building official and possessing knowledge of (a) through (d):

(a) Selection and scaling of ground motions for use in nonlinear response history analysis.
(b) Behavior of structural systems of the type under consideration when subjected to earthquake loading.
(c) Analytical structural modeling for use in nonlinear response history analysis, including use of physical tests in the creation and calibration of the structural analysis models, and knowledge of soil-structure interaction if used in the analysis or in the development of ground motions.
(d) The requirements of Appendix A as they pertain to design of the type of structure under consideration.

A.13.3 The scope of the independent structural design review shall be approved by the building official and shall include a minimum of (a) through (h):

(a) Basis of design document, including the earthquake-performance objectives, the overall earthquake-resistant design methodology, and acceptance criteria
(b) Proposed structural system

COMMENTARY

RA.12.3.3 Analysis of tall buildings with structural core wall systems have shown inelastic response in slabs at their connection to core walls. Integrity of this connection is critical to the overall performance of the structure. Enhanced details, which include properly anchored or continuous reinforcement and post-tensioning tendons, providing additional integrity are required.

RA.12.3.4 Sliding at horizontal construction joints of walls has been observed in earthquakes and in laboratory testing of structural walls. Enhanced detailing is required in regions of high shear to minimize slip or sliding at construction joints.

RA.13—Independent structural design review

RA.13.1 The independent structural design reviewer provides an independent, objective, technical review of those aspects of the structural design of the building that relate to earthquake-performance and advises the building official whether the design meets the acceptance criteria and the expected building performance.

Review by the independent structural design reviewer is not intended to replace quality assurance measures ordinarily exercised by the licensed design professional. Responsibility for the structural design remains solely with the licensed design professional in responsible charge of the structural design.

RA.13.2 On many projects, independent structural design review may be provided by a review team approved by the building official. Each member of the review team may possess specialized knowledge and expertise, and jointly meet the requirements of A.13.2.

An independent structural design reviewer should not have conflicts of interest with respect to the project and should not be part of the design team for the project.

RA.13.3 The scope of the independent structural design review should be clearly defined and acceptable to the building official.

CODE

(c) Earthquake hazard determination, and selection and modification of earthquake ground motions

(d) Modeling approaches for components

(e) Structural analysis model, including soil-structure interaction as applicable, and verification that the structural analysis model adequately represents the properties of the structural system

(f) Review of structural analysis results and determination of whether calculated response meets approved acceptance criteria

(g) Design and detailing of structural components

(h) Drawings, specifications, and quality control/quality assurance and inspection provisions in the design documents

A.13.4 The independent structural design review shall be documented as follows:

(a) The independent structural design reviewer shall issue comments and questions to the licensed design professional.

(b) The licensed design professional shall provide written responses to the independent structural design reviewer.

(c) The independent structural design reviewer shall summarize the review in a letter addressed to the building official that shall include a log of all questions or comments and responses. Any items that lack resolution or consensus shall be clearly explained with reasons for lack of agreement.

COMMENTARY

RA.13.4 A statement of agreement with the design should be provided. However, there may be occasions where complete agreement between the independent structural design reviewer and the licensed design professional cannot be reached. These items should be documented in the summary review letter.

Notes