CHAPTER A5

EARTHQUAKE HAZARD REDUCTION IN EXISTING CONCRETE BUILDINGS AND CONCRETE WITH MASONRY INFILL BUILDINGS

SECTION A501 PURPOSE

The purpose of this chapter is to promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes on concrete buildings and concrete frame buildings with masonry infills.

The provisions of this chapter are intended as minimum standards for structural seismic resistance, and are established primarily to reduce the risk of life loss or injury. Compliance with the provisions in this chapter will not necessarily prevent loss of life or injury or prevent earthquake damage to the rehabilitated buildings.

SECTION A502 SCOPE

The provisions of this chapter shall apply to all buildings having concrete floors or roofs supported by reinforced concrete walls or by concrete frames and columns or to buildings having concrete frames with masonry infill. This chapter shall not apply to buildings with roof diaphragms that are defined as flexible diaphragms by the building code.

Buildings that were designed and constructed in accordance with the seismic provisions of the 1993 *BOCA National Building Code*, the 1994 *Standard Building Code,* the 1976 *Uniform Building Code,* the 2000 *International Building Code* or later editions of these codes shall be deemed to comply with these provisions, unless the seismicity of the region has increased since the design of the building.

Exception: This chapter shall not apply to concrete buildings and concrete with masonry infill buildings where Seismic Design Category A is permitted.

SECTION A503 DEFINITIONS

For the purposes of this chapter, the applicable definitions and notations in the building code and the following shall apply:

MASONRY INFILL. An unreinforced or reinforced masonry wall construction within a reinforced concrete frame.

SEISMIC USE GROUP III. Those buildings categorized as essential facilities or hazardous facilities, or as designated by the building official.

SECTION A504 SYMBOLS AND NOTATIONS

For the purposes of this chapter, the applicable symbols and notations in the building code and the following shall apply.

a_{pi} = Spectral acceleration ordinate of the trial performance point in the Acceleration-Displacement Response Spectra (ADRS) domain.

a_y = Spectral acceleration ordinate of the yield point of the capacity curve in the Acceleration-Displacement Response Spectra (ADRS) domain.

d_{pi} = Spectral displacement ordinate of the trial performance point in the Acceleration-Displacement Response Spectra (ADRS) domain.

d_y = Spectral displacement ordinate of the yield point of the capacity curve in the Acceleration-Displacement Response Spectra (ADRS) domain.

PF_I = Participation factor for the first or primary natural vibration mode of the structure.

S_a = Spectral acceleration.

S_d = Spectral displacement.

SR_A = Modification factor for the 5-percent damped acceleration response spectra in the constant acceleration region.

SR_V = Modification factor for the 5-percent damped acceleration response spectra in the constant velocity region.

V = Total design base shear.

W = Total design seismic dead load as prescribed in the building code.

w_i = Portion of W that is located at or assigned to level i.

a_{LL} = Modal weight coefficient for the first or primary natural vibration mode of the structure.

D_{roof} = Roof displacement relative to the ground.

α_I = Modal weight coefficient for the first or primary natural vibration mode of the structure.

$\phi_{i,I}$ = The first or primary natural vibration mode shape coordinate at floor level i in the direction of the applied seismic loading, S_a.

$\phi_{roof,I}$ = The first or primary natural vibration mode shape coordinate at the roof level in the direction of the applied seismic loading, S_a.

$\phi_{i,j}$ = Displacement amplitude of floor level i in the jth natural vibration mode of the structure.

$\phi_{roof,j}$ = Displacement amplitude of the roof level in the jth natural vibration mode of the structure.

SECTION A505
GENERAL REQUIREMENTS

A505.1 General. This chapter provides a three-tiered procedure to evaluate the need for seismic rehabilitation of existing concrete buildings and concrete buildings with masonry infills. The evaluation shall show that the existing building is in compliance with the appropriate part of the evaluation procedure as described in Sections A507, A508 and A509, or shall be modified to conform to the respective acceptance criteria. This chapter does not preclude a building from being evaluated or modified to conform to the acceptance criteria using other well-established procedures, based on rational methods of analysis in accordance with principles of mechanics and approved by the authority having jurisdiction.

Evaluation of concrete buildings with masonry infill shall be in accordance with Tier 3 analysis procedure as described in Section A509.

A505.2 Properties of in-place materials. Except where specifically permitted herein, the stress-strain relationship of concrete, masonry and reinforcement shall be determined from published data or by testing. All available information, including building plans, original calculations and design criteria, site observations, testing, and records of typical materials and construction practices prevalent at the time of construction, shall be considered when determining material properties.

For Tier 3 analyses, expected material properties shall be used in lieu of nominal properties in the calculation of strength, stiffness and deformabiltity of building components.

The procedure for testing and determination of stress-strain values shall be as prescribed in Sections A505.2.1 through A505.2.5.

A505.2.1 Concrete. The compressive strength of existing concrete shall be determined by tests on cores sampled from the structure.

Exceptions:

1. For Tier 1 analysis, the compressive strength of the concrete may be determined based on the information shown on the original construction documents or based on the values shown in Table A505.1.
2. For Tier 2 analysis, the compressive strength may be determined based on the information shown on the original construction documents.

Core testing shall be performed in accordance with the following:

1. The cutting of cores shall not significantly reduce the strength of the existing structure. Cores shall not be taken in columns. Existing reinforcement shall not be cut.
2. If the construction documents do not specify a minimum compressive strength of the classes of concrete, five cores per story, with a minimum of 10 cores, shall be obtained for testing.

   Exception: If the coefficient of variation of the compressive strength does not exceed 15 percent, the number of cores per story may be reduced to two and the minimum number of tests may be reduced to five.
3. When the construction documents specify a minimum compressive strength, two cores per story per class of concrete shall be taken in the areas where that concrete was to be placed. A minimum of five cores shall be obtained for testing. If a higher strength of concrete was specified for columns than the remainder of the concrete, cores taken in the beams for verification of the specified strength of the beams shall be substituted for tests in the columns. The strength specified for columns may be used in the analyses if the specified compressive strength in the beams is verified.
4. The sampling for the concrete strength tests shall be distributed uniformly in each story. If the building has shear walls, a minimum of 50 percent of the cores shall be taken from the shear walls. Not more than 25 percent of the required cores shall be taken

TABLE A505.1—ASSUMED COMPRESSIVE STRENGTH OF STRUCTURAL CONCRETE (psi)

TIME FRAME	FOOTINGS	BEAMS	SLABS	COLUMNS	WALLS
1965 or earlier	2,000	2,000	2,000	2,000	2,000
1966-Present	3,000	3,000	3,000	3,000	3,000

For SI: 1 pound per square inch = 6.89 kPa.

TABLE A505.2—ASSUMED YIELD STRESS OF EXISTING REINFORCEMENT (psi)

TYPE OF REINFORCEMENT AND ERA	ASSUMED YIELD STRESS (psi)
Pre-1940 structural and intermediate grade, plain or deformed	45,000
Pre-1940 twisted and hard grade	55,000
Post-1940 structural and intermediate grade	45,000
Post-1940 hard grade	60,000
ASTM A 615 Grade 40	50,000
ASTM A 615 Grade 60	70,000

For SI: 1 pound per square inch = 6.89 kPa.

in floor and roof slabs. The remainder of the cores may be taken from the center of beams at mid-span. In concrete frame buildings, 75 percent of the cores shall be taken from the beams.

5. The mean value of the compressive stresses obtained from the core testing for each class of concrete shall be used in the analyses. Values of peak strain that are associated with peak compressive stress may be taken from published data for the nonlinear analyses of reinforced concrete elements.

A505.2.2 Solid-grouted reinforced masonry. The compressive strength of solid-grouted concrete block or brick masonry may be taken as 1,500 pounds per square inch (10.3 MPa). The strain associated with peak stress may be taken as 0.0025.

A505.2.3 Partially grouted masonry. A minimum of five units shall be removed from the walls and tested in conformance with ASTM C 90. Compressive strength of the masonry is permitted to be determined in accordance with Tables 2105.2.2.1.1 and 2105.2.2.1.2 of the *International Building Code*, assuming Type S mortar. The strain associated with peak stress may be taken as 0.0025.

A505.2.4 Unreinforced masonry. The stress-strain relationship of existing unreinforced masonry shall be determined by in-place cyclic testing. The test procedure shall conform to Section A510.

One stress-strain test per story and a minimum of five tests shall be made in the unreinforced masonry infills. The location of the tests shall be uniformly distributed throughout the building.

The average of the stress-strain values obtained from testing shall be used in the nonlinear analyses of frame infill assemblies.

A505.2.5 Reinforcement. The expected yield stress of each type of new or existing reinforcement shall be taken from Table A505.2, unless the reinforcement is sampled and tested for yield stress. The axial reinforcement in columns of post-1933 buildings shall be assumed to be hard grade unless noted otherwise on the construction documents.

A505.3 Structural observation, testing and inspection. Structural observation, in accordance with Section 1709 of the *International Building Code* shall be required for all structures in which seismic retrofit is being performed in accordance with this chapter. Structural observation shall include visual observation of work for conformance with the approved construction documents and confirmation of existing conditions assumed during design.

Structural testing and inspection for new construction materials shall be in accordance with the building code, except as modified by this chapter.

SECTION A506
SITE GROUND MOTION

A506.1 Site ground motion for Tier 1 analysis. The earthquake loading used for the determination of demand on elements of the structure shall correspond to that required by ASCE 31 Tier 1.

A506.2 Site ground motion for Tier 2 analysis. The earthquake loading used for the determination of demand on elements and the structure shall conform to 75 percent of that required by the building code.

A506.3 Site ground motion for Tier 3 analysis. The site ground motion shall be an elastic design response spectrum prepared in conformance with the building code but having spectral acceleration values equal to 75 percent of the code design response spectrum. The spectral acceleration values shall be increased by the occupancy importance factor when required by the building code.

SECTION A507
TIER 1 ANALYSIS PROCEDURE

A507.1 General. Structures conforming to the requirements of the ASCE 31 Tier 1, Screening Phase, are permitted to be shown to be in conformance with this chapter by submission of a report to the building official as described in this section.

A507.2 Evaluation report. The registered design professional shall prepare a report summarizing the analysis conducted in compliance with this section. As a minimum, the report shall include the following items:

1. Building description.
2. Site inspection summary.
3. Summary of reviewed record documents.
4. Earthquake design data used for the evaluation of the building.
5. Completed checklists.
6. Quick-check analysis calculations.
7. Summary of deficiencies.

SECTION A508
TIER 2 ANALYSIS PROCEDURE

A508.1 General. A Tier 2 analysis includes an analysis using the following linear methods: Static or equivalent lateral force procedures. A linear dynamic analysis may be used to determine the distribution of the base shear over the height of the structure. The analysis, as a minimum, shall address all potential deficiencies identified in Tier 1, using procedures specified in this section.

If a Tier 2 analysis identifies a nonconforming condition, such condition shall be modified to conform to the acceptance criteria. Alternatively, the design professional may choose to perform a Tier 3 analysis to verify the adequacy of the structure.

TABLE A508.1—COMPONENT STIFFNESS

COMPONENT	FLEXURAL RIGIDITY	SHEAR RIGIDITY[a]	AXIAL RIGITY
Beam, nonprestressed	$0.3 - 0.5\ E_cI_g$	$0.4\ E_cA_w$	E_cA_g
T- or L-shape beams, nonprestressed	$0.25 - 0.45\ I_g$	$0.4\ E_cA_w$	E_cA_g
Beam, prestressed	$1.0\ E_cI_g$	$0.4\ E_cA_w$	E_cA_g
Column in compression ($P > 0.5\ f'_cA_g$)	$0.7 - 0.9\ E_cI_g$	$0.4\ E_cA_w$	E_cA_g
Column in compression ($P \leq 0.5\ f'_cA_g$)	$0.5 - 0.7\ E_cI_g$	$0.4\ E_cA_w$	E_cA_g
Column in tension	$0.3 - 0.5\ E_cI_g$	$0.4\ E_cA_w$	E_sA_s
Walls	To be determined based on rational procedures	$0.4\ E_cA_w$	E_cA_g
Flat slab, nonprestressed	To be determined based on rational procedures		
Flat slab, prestressed	To be determined based on rational procedures		

a. For shear stiffness, the quantity $0.4\ E_c$ has been used to represent the shear modulus, G.

A508.2 Limitations. A Tier 2 analysis procedure may be used if:

1. There is no in-plane offset in the lateral-force-resisting system.
2. There is no out-of-plane offset in the lateral-force-resisting system.
3. There is no torsional irregularity present in any story. A torsional irregularity may be deemed to exist in a story when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.
4. There is no weak story irregularity at any floor level on any axis of the building. A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

   Exception: Static or equivalent lateral force procedures shall not be used if:

   1. The building is more than 100 feet (30 480 mm) in height.
   2. The building has a vertical mass or stiffness irregularity (soft story). Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of any adjacent story. A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.
   3. The building has a vertical geometric irregularity. Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story.
   4. The building has a nonorthogonal lateral-force-resisting system.

A508.3 Analysis procedure. A structural analysis shall be performed for all structures in accordance with the requirements of the building code, except as modified in Section A506. The response modification factor, R, shall be selected based on the type of seismic-force-resisting system employed and shall comply with the requirements of Section 506.1.1.2.

A508.3.1 Mathematical model. The three-dimensional mathematical model of the physical structure shall represent the spatial distribution of mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its distribution of lateral forces. All concrete and masonry elements shall be included in the model of the physical structure.

Exception: Concrete or masonry partitions that are isolated from the concrete frame members and the floor above.

Cast-in-place reinforced concrete floors with span-to-depth ratios less than three-to-one may be assumed to be rigid diaphragms. Other floors, including floors constructed of precast elements with or without a reinforced concrete topping, shall be analyzed in conformance with the building code to determine if they must be considered semi-rigid diaphragms. The effective in-plane stiffness of the diaphragm, including effects of cracking and discontinuity between precast elements, shall be considered. Parking structures that have ramps rather than a single floor level shall be modeled as having mass appropriately distributed on each ramp. The lateral stiffness of the ramp may be calculated as having properties based on the uncracked cross section of the slab exclusive of beams and girders.

A508.3.2 Component stiffness. Component stiffness shall be calculated based on the approximate values shown in Table A508.1.

A508.4 Design, detailing requirements and structural component load effects. The design and detailing of new components of the seismic-force-resisting system shall comply with the requirements of the *International Building Code*, unless specifically modified herein.

A508.5 Acceptance criteria. The calculated strength of a member shall not be less than the load effects on that member.

A508.5.1 Load combinations. For load and resistance factor design (strength design), structures and all portions thereof shall resist the most critical effects from the combinations of factored loads prescribed in the building code.

Exception: For concrete beams and columns, the shear effect shall be determined based on the most critical load combinations prescribed in the building code. The shear load effect, because of seismic forces, shall be multiplied by a factor of *Cd,* but combined shear load effect need not be greater than *Ve*, as calculated in accordance with Equation (A5-4). M_{pr1} and M_{pr2} are the end moments, assumed to be in the same direction (clockwise or counter clockwise), based on steel tensile stress being equal to 1.25 f_y, where f_y is the specified yield strength.

$$V_e = \frac{M_{pr1} + M_{pr2}}{L} \pm \frac{W_g}{2} \qquad \textbf{(Equation A5-4)}$$

where:

W_g = Total gravity loads on the beam.

A508.5.2 Determination of the strength of members. The strength of a member shall be determined by multiplying the nominal strength of the member by a strength reduction factor, ϕ. The nominal strength of the member shall be determined in accordance with the building code.

SECTION A509
TIER 3 ANALYSIS PROCEDURE

A509.1 General. A Tier 3 evaluation shall be performed using the procedure prescribed in Section A509.2. Alternatively, the procedures prescribed in Sections A509.3 and A509.4 may also be used where specifically permitted herein.

A509.1.1 Mathematical model. The three-dimensional mathematical model of the physical structure shall represent the spatial distribution of mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. All concrete and masonry elements shall be included in the model of the physical structure.

Exception: Concrete or masonry partitions that are isolated from the concrete frame members and the floor above.

Cast-in-place reinforced concrete floors with span-to-depth ratios less than three-to-one may be assumed to be rigid diaphragms. Other floors, including floors constructed of precast elements with or without a reinforced concrete topping, shall be analyzed in conformance with the building code to determine if they must be considered semi-rigid diaphragms. The effective in-plane stiffness of the diaphragm, including effects of cracking and discontinuity between precast elements, shall be considered. Parking structures that have ramps rather than a single floor level shall be modeled as having mass appropriately distributed on each ramp. The lateral stiffness of the ramp may be calculated as having properties based on the uncracked cross section of the slab exclusive of beams and girders.

A509.1.2 Acceptance criteria.

A509.1.2.1 Compressive strain determination. Compressive strain in columns, shear walls and infills may be determined by the nonlinear analysis or a procedure that assumes plane sections remain plane.

Compressive strain shall be determined for combined flexure and axial loading. The seismic flexural moments and axial load shall be taken from the response spectrum analysis for frame or shear wall buildings, and from the substructure model for infill frames. The combination of critical effects for analysis of compressive strain shall be those given in the building code for strength design or load and factor resistance design.

A509.1.2.2 Story drift limitation. Story drift is the displacement of one level relative to the level above or below, calculated by the response spectrum analysis using the appropriate effective stiffness. The story drift is limited to displacement that causes any of the following effects:

1. Compressive strain of 0.003 in the frame confining infill or in a shear wall.
2. Compressive strain of 0.004 in a reinforced concrete column, unless the engineer can show by published experimental research that the existing confinement reinforcement justifies higher values of strain.
3. Peak strain in masonry infills as determined by experimental data or by physical testing as prescribed in Section A510.
4. Displacement that was calculated by the nonlinear analysis as to when strength degradation of any element began.

Exception: Item 4 may be taken as the displacement that causes a strength degradation in that line of resistance equal to 10 percent of the sum of the strength of the elements in that line of resistance.

A509.1.2.3 Shear strength limitation. The required in-plane shear strength of all columns, piers and shear walls shall be the shear associated with the moments induced at the ends of columns or piers and at the base of shear walls by the story displacements. No strength reduction factors shall be used in the determination of strength.

A509.2 Pseudo-nonlinear dynamic analysis procedure. Structures shall be analyzed for seismic forces acting concurrently on the orthogonal axes of the structure. The effects of the loading on two orthogonal axes shall be combined by SRSS methods. The analysis shall include all torsional effects. Accidental torsional effects need not be considered.

A509.2.1 Determination of the effective stiffness.

A509.2.1.1 General. The effective stiffness of concrete and masonry elements or systems shall be calculated as the secant stiffness of the element or system with due consideration of the effects of tensile cracking and compression strain. The secant stiffness shall be taken from the force-displacement relationship of the element or system. The secant stiffness shall be measured as the slope from the origin to the intersection of the force-displacement relationship at the assumed displacement. The force-displacement relationship shall be determined by a nonlinear analysis. The force-displacement analysis shall include the calculation of the displacement at which strength degradation begins.

Exception: The initial effective moment of inertia of beams and columns in shear wall or infilled frame buildings may be estimated using Table A508.1. The ratio of effective moment of inertia used for the beams and for the columns shall be verified by Equations (A5-5), (A5-6) and (A5-7). The estimates shall be revised if the ratio used exceeds the ratio calculated by more than 20 percent.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

(Equation A5-5)

where:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \textbf{(Equation A5-6)}$$

and

$$f_r = 7.5\sqrt{f'_c} \qquad \textbf{(Equation A5-7)}$$

A509.2.1.2 Effective stiffness of infills. The effective stiffness of an infill shall be determined from a nonlinear analysis of the infill and the confining frame. The effect of the infill on the stiffness of the system shall be determined by differentiating the force-displacement relationship of the frame-infill system from the frame-only system.

A509.2.1.3 Model of infill. The mathematical model of an infilled frame structure shall include the stiffness effects of the infill as a pair of diagonals in the bays of the frame. The diagonals shall be considered as having concrete properties and only axial loads. Their lines of action shall intersect the beam-column joints. The secant stiffness of the force-displacement relationship, calculated as prescribed in Section A509.2.1.2, shall be used to determine the effective area of the diagonals. The effective stiffness of the frame shall be determined as specified in Section A509.2.1.1. Other procedures that provide the same effective stiffness for the combination of infill and frame may be used when approved by the building official.

A509.2.2 Description of analysis procedures. The pseudo-nonlinear dynamic analysis is an iterative response spectrum analysis procedure using effective stiffness as the stiffness of the structural components. The response spectrum analysis shall use the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve that corresponds to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

The effective stiffnesses shall be determined by an iterative method. The mathematical model using assumed effective stiffnesses shall be used to calculate dynamic displacements. The effective stiffness of all concrete and masonry elements shall be modified to represent the secant stiffness obtained from the nonlinear force-displacement analysis of the element or system at the calculated displacement. A re-analysis of the mathematical model shall be made using the adjusted effective stiffness of existing and supplemental elements and systems until closure of the iterative process is obtained. A difference of 10 percent from the effective stiffness used and that recalculated may be assumed to constitute closure of the iterative process.

A509.2.2.1 Number of modes. At least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

A509.2.2.2 Combining modes. The peak displacements for each mode shall be combined by recognized methods. Modal interaction effects of three-dimensional models shall be considered when combining modal maxima.

A509.3 Capacity spectrum analysis procedure.

A509.3.1 General. This section presents an alternative procedure for a nonlinear static analysis for verification of acceptable performance by comparing the available capacity to the earthquake demand.

Where inelastic torsional response is a dominant feature of overall response, the engineer shall use either a retrofit that reduces the torsional response or an alternative analysis procedure. Inelastic torsional response may be deemed to exist if torsional irregularity as defined in Section A508.2 is present in any story.

The behavior of foundation components and the effects of soil-structure interaction shall be modeled or shown to be insignificant to building response.

A509.3.2 Modeling of building components.

A509.3.2.1 Component initial stiffness. Component initial stiffness shall be represented by a secant value defined by the effective yield point of the component.

The effective initial stiffness shall be calculated using principles of mechanics, with due consideration of the effects of tensile cracking and compression strain.

Exception: Component effective initial stiffness may be calculated using the approximate values shown in Table A508.1.

A509.3.2.2 Component strength. The strength of building components shall be calculated using the procedures outlined in the appropriate section of the building code.

Exception: Component properties may be calculated using the principles of mechanics as verified by experimental results.

A509.3.2.3 Component deformability. The deformability of building components shall be obtained from nonlinear load-deformation relationships that are appropriate for the component being considered. The nonlinear load-deformation relationship shall include information on the plastic deformation capacity at which lateral strength degrades, the plastic deformation capacity at which gravity-load resistance degrades, and the residual strength of the component after strength degradation.

The nonlinear load-deformation relationships of building components shall be determined from nonlinear analyses based on the principles of mechanics, experimental data or established values published in technical literature, as approved by the building official.

A509.3.3 Description of analysis procedures.

A509.3.3.1 Determination of the capacity curve. The structure's capacity shall be represented by a capacity curve, which is a plot of the building's base shear versus roof displacement. The capacity curve shall be determined by performing a series of sequential analyses with increasing lateral load, using a mathematical model that accounts for reduced resistance of yielding components. The analysis should include the effect of gravity loads on the building's response to lateral loads.

Lateral forces shall be applied to the structure in proportion to the product of mass and fundamental mode shape.

Exceptions:

1. For buildings with weak stories, the vertical distribution of lateral forces shall be modified to reflect the changed fundamental mode shape after yielding of the weak story.
2. For buildings over 100 feet (30 480 mm) in height or buildings with irregularities that cause significant participation from modes of vibration other than the fundamental mode, the vertical distribution of lateral forces shall reflect the contribution of higher modes.

A509.3.3.2 Conversion of the capacity curve to the capacity spectrum. The capacity curve calculated in Section A509.3.3.1 shall be converted to the capacity spectrum, which is a representation of the capacity curve in the Acceleration-Displacement Response Spectra (ADRS) format. Each point on the capacity curve shall be converted using Equations (A5-8) and (A5-9).

$$a = \frac{V / W}{\alpha_1} \qquad \textbf{(Equation A5-8)}$$

$$d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}} \qquad \textbf{(Equation A5-9)}$$

where:

$$PF_1 = \frac{\sum_{i=1}^{N} W_i \phi_{i1} / g}{\sum_{i=1}^{N} W_i \phi_{i1}^2 / g} \qquad \textbf{(Equation A5-10)}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} W_i \phi_{i1} / g\right]^2}{\left[\sum_{i=1}^{N} W_i / g\right]\left[\sum_{i=1}^{N} W_i \phi_{i1}^2 / g\right]} \qquad \textbf{(Equation A5-11)}$$

A509.3.3.3 Bilinear representation of the capacity spectrum. A bilinear representation of the capacity spectrum curve obtained in Section A509.3.3.2 shall be used in estimating the appropriate reduction of spectral demand. The first segment of the bilinear representation of the capacity spectrum shall be a line from the origin at the initial stiffness of the building using the component initial stiffness specified in Table A508.1. The second segment of the bilinear representation of the capacity spectrum shall be a line back from the trial performance point, a_{pi}, d_{pi}, at a slope that results in the area under the bilinear representation being approximately equal to the area under the actual capacity spectrum curve. The intersection of the two segments of the bilinear representation of the capacity spectrum shall determine the yield point a_y, d_y.

A509.3.3.4 Development of the demand spectrum. The demand spectrum is a plot of the spectral acceleration and spectral displacement of the demand earthquake ground motion in the Acceleration-Displacement Response Spectra (ADRS) format. The 5-percent damped acceleration response spectra in Section A506 shall be modified for use in the capacity spectrum analysis procedure as follows:

1. In the constant acceleration region, the 5-percent damped acceleration spectra shall be multiplied by:

$$SR_A = 1.51 - 0.32 \ln\left[\frac{21(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right] \geq 0.56$$

(Equation A5-12)

2. In the constant velocity region, the 5-percent damped acceleration spectra shall be multiplied by:

$$SR_v = 1.40 - 0.25 \ln\left[\frac{21(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right] \geq 0.67$$

(Equation A5-13)

3. The spectral displacement ordinate, S_d, for a corresponding spectral acceleration, S_a, shall be determined from:

$$S_d = S_a \left(\frac{T}{2\pi}\right)^2 g \qquad \textbf{(Equation A5-14)}$$

A509.3.3.5 Calculation of the performance point. The performance point shall represent the maximum roof displacement expected for the demand earthquake ground motion. When the displacement of intersection of the capacity spectrum defined in Section A509.3.3.2 and the demand spectrum defined in Section A509.3.3.4 is within 5 percent of the displacement of the trial performance point, a_{pi}, d_{pi}, used in Section A509.3.3.3, the trial performance point shall be considered the performance point. If the intersection of the capacity spectrum and the demand spectrum is not within the acceptable tolerance of 5 percent, a new trial performance point shall be selected and the analysis shall be repeated.

A509.3.4 Response limits. The inter-story drift between floors of the building and the corresponding strains in building components shall be checked at the performance point to verify acceptability under the demand earthquake ground motion. Performance shall be considered acceptable if building response parameters do not exceed the limitations outlined in Section A509.1.2.

A509.4 Displacement coefficient analysis procedure.

A509.4.1 General. This section presents a procedure for generalized nonlinear static analysis for verification of acceptable performance by comparing the available capacity to the earthquake demand.

Where inelastic torsional response is a dominant feature of overall response, the engineer shall use either a retrofit that reduces the torsional response or an alternative analysis procedure. Inelastic torsional response may be deemed to exist if there is torsional irregularity as defined in Section A508.2 present in any story.

The mathematical model of the building shall be determined in accordance with Section A509.1. The general procedure for execution of the displacement coefficient analysis shall be determined in accordance with Section A509.4.5.

Results of the displacement coefficient analysis procedure shall be checked using the applicable acceptance criteria specified in Section A509.1.2.

For three-dimensional analyses, the static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each story level. Effects of accidental torsion shall be considered.

For two-dimensional analyses, the mathematical model describing the framing along each axis of the building shall be developed. The effects of horizontal torsion shall be considered by increasing the target displacement (see Section A509.4.2) by a displacement multiplier, η. The displacement multiplier is the ratio of the maximum displacement at any point on any floor diaphragm (including torsional effects for actual torsion and accidental torsion) to the average displacement on that diaphragm.

The behavior of foundation components and effects of soil-structure interaction shall be modeled or shown to be insignificant to building response.

A509.4.2 Target displacement (δ_t). The target displacement of the control node (typically the center of mass of the building's roof) shall be determined using the following equation:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

where:

C_0 = Modification factor to relate spectral displacement to expected building roof displacement. Value of C_0 can be estimated using any one of the following:

1. The first modal participation factor at the level of the control node.
2. The modal participation factor at the level of the control node computed using a shape vector corresponding to the deflected shape of the building at the target displacement.
3. The appropriate value from Table A509.4.2.

TABLE A509.4.2—VALUES OF MODIFICATION FACTOR, C_0

NUMBER OF STORIES	C_0
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

Linear interpolation shall be used to calculate intermediate values.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements for linear elastic response. C_1 shall not be taken as less than 1.0.

= 1.0 for $T_e \geq T_0$

= $[1.0 + (R - 1)T_0/T_e]/R$ for $T_e < T_0$

where:

R = Strength ratio = $\dfrac{S_a}{V_y / W} \dfrac{1}{C_0}$

V_y = Yield strength calculated using the results of static pushover analysis where the nonlinear base-shear roof-displacement curve of the building is characterized by a bilinear relation (see Section A509.4.5).

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

C_2 = Modification factor to represent the effect of hysteresis shape on maximum displacement response.

= 1.3 where $T > T_0$

= 1.1 where $T \geq T_0$

Exception: Where the stiffness of the structural component in a lateral-force-resisting system, which resists no less than 30 percent of the story shear, does not deteriorate at the target displacement level, C_2 may be assumed to be equal to 1.0.

Sa = Response spectral acceleration at the effective fundamental period and damping ratio of the building, g, in the direction under consideration.

Te = Effective fundamental period of the building in the direction under consideration, per Section A509.4.5.

A509.4.3 Lateral load patterns. Two different vertical distributions of loads shall be used. The first load pattern, termed as the uniform pattern, shall be based on lateral forces proportional to the mass at each story level. The second pattern, called the modal pattern, shall be selected from one of the following:

1. A lateral load pattern represented by C_{vx}, if more than 75 percent of mass participates in the fundamental mode in the direction under consideration. C_{vx} is given by the following expression:

$$C = \frac{w_x h^k}{\sum_{j=1}^{n} wh_i}$$

where:

w_i = Portion of the total building weight, W, located on or assigned to floor level i.

h_i = Height in feet from base to floor level i.

w_x = Portion of the total building weight, W, located on or assigned to floor level x.

h_x = Height in feet from base to floor level x.

k = 1.0 for $T_e \leq 0.5$ sec.

= 2.0 for $T_e \geq 2.5$ sec.

Linear interpolation shall be used to estimate k for intermediate values of T_e.

2. A lateral load pattern proportional to the story inertia forces consistent with the story shear distribution computed by combination of modal responses using response spectrum analysis of the building, including a sufficient number of modes to capture 90 percent of the total seismic mass and the appropriate ground motion spectrum.

A509.4.4 Period determination. The effective fundamental period, T_e, in the direction under consideration, shall be determined using the force-displacement relation of the nonlinear static pushover analysis. The nonlinear relation between the base shear and target displacement of the control node shall be replaced by a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building. The effective lateral stiffness shall be taken as the secant stiffness calculated at a base shear force equal to 60 percent of the yield strength. The effective fundamental period, T_e, shall then be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$

where:

T_i = Elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis.

K_i = Elastic lateral stiffness of the building in the direction under consideration.

K_e = Effective lateral stiffness of the building in the direction under consideration.

A509.4.5 General execution procedure for the displacement coefficient analysis procedure. The general procedure for the execution of the displacement coefficient analysis procedure shall be as follows:

1. An elastic structural model shall be created that includes all components (existing and new) contributing significantly to the weight, strength, stiffness or stability of the structure, and whose behavior is important in satisfying the intended seismic performance.
2. The structural model shall be loaded with gravity loads before application of the lateral loads.
3. The mathematical model shall be subjected to incremental lateral loads using one of the lateral load patterns described in Section A509.4.3. At least two different load patterns shall be used in each principal direction.
4. The intensity of the lateral load shall be monotonically increased until the weakest component reaches a deformation at which there is a significant change in its stiffness. The stiffness properties of this "yielded" component shall be modified to reflect the post-yield behavior, and the modified structure shall be subjected to an increase in lateral loads (for load control) or displacements (for displacement control) using the same lateral load pattern.
5. The previous step shall be repeated as more components reach their yield strengths. At each stage, the internal forces and deformations (both elastic and plastic) of all components shall be computed.
6. The forces and deformations from all previous loading stages shall be accumulated to obtain the total force and deformations of all components at all stages.
7. The loading process shall be continued until unacceptable performance is detected or until a roof displacement is obtained that is larger than the

maximum displacement expected in the design earthquake at the control node.

8. A plot of the control node displacement versus base shear at various stages shall be created. This plot is indicative of the nonlinear response of the structure, and changes in the slope of this load-displacement curve are indicative of the yielding of various components.
9. The load-displacement curve obtained in Item 8 shall be used to compute the effective period of the structure, which would then be used to estimate the target displacement (Section A509.4.2).
10. Once the target displacement has been determined, the accumulated forces and deformations at this displacement shall be used to evaluate the performance of various components.
11. If either the force-demands in the nonductile components or deformation-demands in the ductile components exceed the permissible values, then the component shall be deemed to violate the performance criterion, indicating that rehabilitation be performed for such elements.

The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150 percent of the target displacement, δ_t.

A509.4.6 Acceptance criteria. The inter-story drift between floors of the building and the corresponding strains in building components shall be checked at 150 percent of the target displacement, δ_t, to verify acceptability under the demand earthquake ground motion. Performance shall be considered acceptable if building response parameters do not exceed the limitations outlined in Section A509.1.2.

Exception: Where the effective stiffness, K_e, and the yield strength, V_y, of the building can be determined through rational analysis, the acceptance criteria may be determined based on 100 percent of the target displacement, δ_t.

SECTION A510
DETERMINATION OF THE STRESS-STRAIN RELATIONSHIP OF EXISTING UNREINFORCED MASONRY

A510.1 Scope. This section covers procedures for determining the expected compressive modulus, peak strain and peak compressive stress of unreinforced brick masonry used for infills in frame buildings.

A510.2 General procedure. The outer wythe of multiple wythe brick masonry shall be tested by inserting two flat jacks into the mortar joints of the outer wythe. The prism height (the vertical distance between the flat jacks) shall be five bricks high. The test location shall have adequate overburden and/or vertical confinement to resist the flat jack forces.

A510.3 Preparation for the test. Remove a mortar joint at the top and bottom of the test prism by saw-cutting or drilling and grinding to a smooth surface. The cuts for inserting the flat jacks shall not have a deviation from parallel of more than $^3/_8$ inch (9.53 mm). The deviation from parallel shall be measured at the ends of the flat jacks. The width of the saw cut shall not exceed the width of the mortar joint. The length of the saw cut on the face of the wall may exceed the length of the flat jacks by not more than twice the thickness of the outer wythe plus 1 inch (25.4 mm).

A510.4 Required equipment. The flat jacks shall be rectangular or with semi-circular ends to mimic the radius of the saw blade used to cut the slot for the flat jack. The length of the flat jack shall be 18 inches (457 mm) maximum and 16 inches (406 mm) minimum. This length shall be measured on the longest edge of a flat jack with semi-circular ends. The maximum width of the flat jack shall not exceed the average width of the wythe of brick that is loaded. The minimum width of a flat jack shall be $3^1/_2$ inches (89 mm) measured out-to-out of the flat jack. The flat jack shall have a minimum of two ports to allow air in the flat jack to be replaced by hydraulic fluid. The unused port shall be sealed after all of the air is forced out of the flat jack. The thickness of the flat jack shall not exceed three-quarters of the minimum height of the mortar joint. It is recommended that the height of the flat jack be about one-half of the width of the slot cut for installation of the flat jack. The remaining space can be filled with steel shim plates having plan dimensions equal to the flat jack.

A510.5 Data acquisition equipment. The strain in the tested prism shall be recorded by gauges or similar recording equipment having a minimum range of 0.0001 inch (0.0025 mm). The compressive strain shall be measured on the surface of the prism and shall have a gauge length, measured vertically on the face of the prism, of 10 inches (254 mm) minimum. The gauge points shall be fixed to the wall by drilled-in anchors or by anchors set in epoxy or similar material. The support for the data-recording apparatus shall be isolated from the wall by a minimum of $^1/_{16}$ inch (1.5 mm), so that the gauge length used in the calculation of strain can be taken as the measured length between the anchors of the equipment supports. The gauging equipment shall be as close to the face of the prism as possible, to minimize the probability of erroneous strain measurements caused by bulging of the prism outward from its original plane.

The compressive strain data shall be measured at a minimum of two points on the vertical face of the prism. These points shall be the one-third points of the length of the flat jacks plus or minus $^1/_2$ inch (12.7 mm). As an alternative, the strain may be measured at three points on the face of the prism.

These points shall be spaced at one-quarter of the flat jack length plus or minus $^1/_2$ inch (12.7 mm).

A horizontal gauge at midheight of the prism may be used to record Poisson strain, but this recording data should be considered secondary in importance to the vertical gauges, and the horizontal gauge's placement shall not interfere with placing the vertical gauging as close as possible to the face of the prism.

A510.6 Loading and recording data. The loading shall be applied by hydraulic pumps that add hydraulic fluid to the flat jacks in a controlled method. The application of load shall be incremental and held constant while strains are recorded. The increasing loading for each cycle of loading shall be divided into a minimum of four equal load increments. The strain shall be recorded at each load step. The decrease in loading shall be divided into a minimum of two equal unloading increments. Strain shall be recorded on the decreasing load steps. The hydraulic pressure shall be reduced to zero, and the permanent strain caused by this cycle of loading shall be recorded. This procedure shall be used for each cycle of loading.

The load applied in each cycle of loading shall be determined by estimating the peak compressive stress of the existing brick masonry. The hydraulic pressure needed to cause this peak compressive stress in the prism shall be calculated by assuming that the area of the loaded prism is equal to the area of the flat jack. A maximum of one-third of this pressure, rounded to the nearest 25 pounds per square inch (172 kPa), shall be applied in the specified increments to the peak pressure prescribed for the first cycle of loading. After recording the strain data, this pressure shall be reduced in a controlled manner, for each of the specified increments for unloading and for recording data. The maximum jack pressure on the subsequent cycles shall be one-half, two-thirds, five-sixths and estimated peak pressure. If the estimated peak compressive stress is less than the existing peak compressive stress, the cyclic loading and unloading shall continue using increments of increasing pressure equal to those used prior to the application of estimated peak pressure.

All strain data shall be recorded to 0.0001 inch (0.0025 mm). Jack pressure shall be recorded in increments of 25 pounds per square inch (172 kPa) pressure.

A510.7 Quality control. The flat jack shall be calibrated before use by placing the flat jack between bearing plates of 2-inch (51 mm) minimum thickness in a calibrated testing machine. A calibration curve to convert hydraulic pressure in the flat jack to total load shall be prepared and included in the report of the test results. Flat jacks shall be recalibrated after three uses.

The hydraulic pressure in the flat jacks shall be determined by a calibrated dial indicator having a subdivision of 25 pounds per square inch (172 kPa) or less. The operator of the hydraulic pump shall use this dial indicator to control the required increments of hydraulic pressure in loading and unloading.

A510.8 Interpretation of the data. The data obtained from the testing required by Section A505.2.4 shall be averaged both in the expected peak compressive stress and the corresponding peak strain. The envelope of the averaged stress-strain relationship of all tests shall be used for the material model of the masonry in the infilled frame. If two strain measurements have been made on the surface of the prism, these strain measurements shall be averaged for determination of the stress-strain relationship for the test. If three strain measurements have been made on the surface of the prism, the data recorded by the center gauge shall be given a weight of two for preparing the average stress-strain relationship for the test.

TABLE A5-A—BASIC STRUCTURAL CHECKLIST

This basic structural checklist shall be completed when required by Section A507 prior to completing the corresponding supplemental structural checklist.

Each of the evaluation statements on this checklist shall be marked compliant (C), noncompliant (NC), or not applicable (N/A) for a Tier I evaluation. Compliant statements identify issues that are acceptable according to the criteria of this chapter and the building code, while noncompliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. Noncompliant items shall be mitigated by rehabilitating the structure, or shall be shown to be compliant by performing a Tier 2 or Tier 3 analysis.

BUILDING SYSTEM **General**			
C	NC	N/A	LOAD PATH: The structure shall contain one complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.
C	NC	N/A	ADJACENT BUILDINGS: An adjacent building shall not be located next to the structure being evaluated closer than 4 percent of the height.
C	NC	N/A	MEZZANINES: Interior mezzanine levels shall be braced independently of the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.
Configuration			
C	NC	N/A	WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story above or below.
C	NC	N/A	SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the stiffness in an adjacent story above or below, or less than 80 percent of the average stiffness of the three stories above or below.
C	NC	N/A	GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories, excluding one-story penthouses.
C	NC	N/A	VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation.
C	NC	N/A	MASS: There shall be no change in effective mass of more than 50 percent from one story to the next.
C	NC	N/A	TORSION: The distance between the story center of mass and the story center of regidity shall be less than 20 percent of the building width in either plan dimension.
Condition of Materials			
C	NC	N/A	DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements.
C	NC	N/A	POST-TENSIONING ANCHORS: There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used.
C	NC	N/A	MASONRY UNITS: There shall be no visible deterioration of masonry units.
C	NC	N/A	MASONRY JOINTS: The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.
C	NC	N/A	CONCRETE WALL CRACKS: All existing diagonal cracks in wall elements shall be less than $^1/_8$ inch for Seismic Use Groups other than Group III and less than $^1/_{16}$ inch for Seismic Use Group III; shall not be concentrated in one location; and shall not form a X pattern.
C	NC	N/A	REINFORCED MASONRY WALL CRACKS: All existing diagonal cracks in wall elements shall be less than $^1/_8$ inch for Seismic Use Groups other than Group III and less than $^1/_{16}$ inch for Seismic Use Group III; shall not be concentrated in one location; and shall not form a X pattern.
C	NC	N/A	CRACKS IN BOUNDARY COLUMNS: There shall be no existing diagonal cracks wider $^1/_8$ inch for Seismic Use Groups other than Group III and wider than $^1/_{16}$ inch for Seismic Use Group III in concrete colums that encase masonry infills.
LATERAL-FORCE-RESISTING SYSTEM **Moment Frames** **General**			
C	NC	N/A	REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to two for Seismic Use Groups I, II and III. The number of bays of moment frames in each line shall be greater than or equal to two for Seismic Use Groups other than Group III, and three for Seismic Use Group III.
Moment Frames with Infill Walls			
C	NC	N/A	INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.

(Continued)

TABLE A5-A—BASIC STRUCTURAL CHECKLIST—continued

Concrete Moment Frames			
C	NC	N/A	SHEAR STRESS CHECK: The sear stress in the concrete columns, calculated using the quick-check procedure of Section A507.6.1, shall be less than 100 psi or $2\sqrt{f'_c}$.
C	NC	N/A	AXIAL STESS CHECK: The axial stress because of gravity loads in columns subjected to overturning forces shall be less than $0.10 \times f'_c$. Alternatively, the axial stresses because of overturning forces alone, calculated using the quick-check procedure of Section A507.6.3, shall be less than $0.30 \times f'_c$.
Frames Not Part of the Lateral-Force-Resisting System			
C	NC	N/A	COMPLETE FRAMES: Steel or concrete frames classified as nonlateral-force-resisting components shall form a complete vertical-load-carrying syste.
Shear Walls **General**			
C	NC	N/A	REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to two.
Concrete Shear Walls			
C	NC	N/A	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the quick-check procedure of Section A507.6.2, shall be less than 100 psi or $2\sqrt{f'_c}$.
C	NC	N/A	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be greater than 0.0015 in vertical direction and greater than 0.0025 in the horizontal direction. The spacing of reinforcing steel shall be equal to or less than 18 inches.
CONNECTIONS **Shear Transfer**			
C	NC	N/A	TRANSFER TO SHEAR WALLS: The diaphragm shall be reinforced and connected for transfer of loads to the shear walls for Seismic Use Groups I and II, and the connections shall be able to develop the shear strength of the walls for Seismic Use Group III.
Vertical Components			
C	NC	N/A	CONCRETE COLUMNS: All concrete columns shall be doweled into the foundation for Seismic Use Groups I and II, and the dowels shall be able to develop the tensile capacity of the column for Seismic Use Group III.
C	NC	N/A	WALL REINFORCING: Walls shall be doweled into the foundation for Seismic Use Groups I and II, and the dowels shall be able to develop the strength of the walls for Seismic Use Group III.

For SI: 1 inch = 25.4 mm.

TABLE A5-B—SUPPLEMENTAL STRUCTURAL CHECKLIST

This supplemental structural checklist shall be completed when required by Section A507. The basic structural checklist shall be completed prior to completing this supplemental structural checklist.			
LATERAL-FORCE-RESISTING SYSTEM **Moment Frames** **Concrete Moment Frames**			
C	NC	N/A	FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.
C	NC	N/A	PRESTRESSED FRAME ELEMENTS: The lateral-load-resisting frames shall not include any prestressed or post-tensioned elements.
C	NC	N/A	SHORT CAPTIVE COLUMNS: There shall be no columns at a level with height-depth ratios less than 50 percent of the nominal height-depth ratio of the typical columns at that level for Seismic Use Groups other than Group III, and less than 75 percent for Seismic Use Group III.
C	NC	N/A	NO SHEAR FAILURES: The shear capacity of frame members shall be able to develop the moment capacity at the top and bottom of the columns.
C	NC	N/A	STRONG COLUMN/WEAK BEAM: The sum of the moment capacity of the columns shall be 20-percent greater than that of the beams at frame joints.
C	NC	N/A	BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25 percent of the longitudinal bars provided at the joints for either positive or negative moment shall be continuous throughout the length of the members.
C	NC	N/A	COLUMN-BAR SPLICES: All column bar lap-splice lengths shall be greater than 35 d_b for Seismic Use Groups other than Group III and greater than 50 d_b for Seismic Use Group III, and shall be enclosed by ties spaced at or less than 8 d_b for all Seismic Use Groups.
C	NC	N/A	BEAM-BAR SPLICES: The lap splices for longitudinal beam reinforcing shall not be located within $l_b/4$ of the joints and shall not be located within the vicinity of potential plastic hinge locations.
C	NC	N/A	COLUMN-TIE SPACING: Frame columns shall have ties spaced at or less than $d/4$ throughout their length and at or less than 8 d_b at all potential plastic hinge locations.
C	NC	N/A	STIRRUP SPACING: All beams shall have stirrups spaced at or less than d/2 throughout their length. At potential plastic hinge locations, stirrups shall be spaced at or less than the minimum of 8 d_b or $d/4$.
C	NC	N/A	JOINT REINFORCING: Beam-column joints shall have ties spaced at or less than 8 d_b.
C	NC	N/A	JOINT ECCENTRICITY: For Seismic Use Group III, there shall be no eccentricities larger than 20 percent of the smallest column plan dimension between girder and column centerlines.
C	NC	N/A	STIRRUP AND TIE HOOKS: For Seismic Use Group III, the beam stirrups and column ties shall be anchored into the member cores with hooks of 135° or more.
Frames Not Part of the Lateral-Force-Resisting System			
C	NC	N/A	DEFORMATION COMPATIBILITY: Nonlateral-force-resisting components shall have the shear capacity to develop the flexural strength of the elements for Seismic Use Groups other than Group III and shall have ductile detailing for Seismic Use Group III.
C	NC	N/A	FLAT SLABS: Flat slabs/plates classified as nonlateral-force-resisting components shall have continuous bottom steel through the column joints for Seismic Use Groups other than Group III. Flat slabs/plates shall not be permitted for Seismic Use Group III.
Shear Walls **Concrete Shear Walls**			
C	NC	N/A	COUPLING BEAMS: The stirrups in all coupling beams over means of egress shall be spaced at or less than $d/2$ and shall be anchored into the core with hooks of 135° or more. In addition, the beams shall have the capacity in shear to develop the uplift capacity of the adjacent wall for Seismic Use Group III.
C	NC	N/A	OVERTURNING: For Seismic Use Group III, all shear walls shall have aspect ratios less than 4:1. Wall piers need not be considered.
C	NC	N/A	CONFINEMENT REINFORCING: For shear walls in Seismic Use Group III with aspect ratios greater than 2.0, boundary elements shall be confined with spirals or ties with spacing less than 8 d_b.
C	NC	N/A	REINFORCING AT OPENINGS: For Seismic Use Group III, there shall be added trim reinforcement around all wall openings.
C	NC	N/A	WALL THICKNESS: For Seismic Use Group III, thickness of bearing walls shall not be less than $^1/_{25}$ the minimum unsupported height or length, or less than 4 inches.

(Continued)

TABLE A5-B—SUPPLEMENTAL STRUCTURAL CHECKLIST—continued

DIAPHRAGMS **General**			
C	NC	N/A	DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors.
C	NC	N/A	DIAPHRAGM OPENINGS ADJACENT TO SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25 percent of the wall length for Seismic Use Groups I and II, and less than 15 percent of the wall length for Seismic Use Group III.
C	NC	N/A	PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to Seismic Use Group III only.
C	NC	N/A	DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcement around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to Seismic Use Group III only.
Other Diaphragms			
C	NC	N/A	OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than those described in Section A502.
CONNECTIONS **Vertical Components**			
C	NC	N/A	LATERAL LOAD AT PILE CAPS: Pile caps shall have top reinforcement, and piles shall be anchored to the pile caps for Seismic Use Groups I and II. The pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Seismic Use Group III.

For SI: 1 inch = 25.4 mm.

TABLE A5-C—GEOLOGIC SITE HAZARD AND FOUNDATION CHECKLIST

This geologic site hazard and foundation checklist shall be completed when required by Section A507. Each of the evaluation statements on this checklist shall be marked compliant (C), noncompliant (NC), or not applicable (N/A) for a Tier I evaluation. Compliant statements identify issues that are acceptable according to the criteria of this chapter and the building code, while noncompliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. Noncompliant items shall be mitigated by rehabilitating the structure, or shall be shown to be compliant by performing a Tier 2 or Tier 3 analysis.			
Geologic Site Hazards			
The following statements shall be completed building in regions of high or moderate seismicity:			
C	NC	N/A	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building for Seismic Use Groups I, II, and III.
C	NC	N/A	SLOPE FAILURE: The building site shall either be sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures, or shall be capable of accommodating any predicted movements without failure.
C	NC	N/A	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.
Condition of Foundations			
The following statement shall be completed for all Tier I building evaluations.			
C	NC	N/A	FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure.
The following statement shall be completed for buildings in regions of high or moderate seismicity being evaluated to Seismic Use Group III:			
C	NC	N/A	DETERIORATION: There shall not be evidence that foundation elements have deteriorated because of corrosion, sulfate attack, material breakdown or other reasons in a manner that would affect the integrity or strength of the structure.
Capacity of Foundations			
The following statement shall be completed for all Tier I building evaluations.			
C	NC	N/A	POLE FOUNDATIONS: Pole foundations shall have a minimum embedment depth of 4 feet for Seismic Use Groups I, II, and III.
The following statements shall be completed for buildings in regions of high seismicity and for buildings in regions of moderate seismicity evaluated to Seismic Use Group III:			
C	NC	N/A	OVERTURNING: The ratio of the effective horizontal dimension at the foundation level of the lateral-force-resisting system to the building height (base/height) shall be greater than $0.6S_a$.
C	NC	N/A	TIES BETWEEN FOUNDATION ELEMENTS: The foundation shall have ties adequate to resist seismic forces where footings, piles and piers are not restrained by beams, slabs or soils classified as Class A, B or C.
C	NC	N/A	DEEP FOUNDATIONS: Piles and piers shall be capable of transferring the lateral forces between the structure and the soil. This statement shall apply to Seismic Use Group III only.
C	NC	N/A	SLOPING SITES: The grade difference from one side of the building to another shall not exceed one-half the story height at the location of embedment. This statement shall apply to Seismic Use Group III Performance Level only.

For SI: 1 foot = 304.8 mm.