APPENDIX A
STRENGTH DESIGN OF AUTOCLAVED AERATED CONCRETE (AAC) MASONRY

A.1 — General

A.1.1 *Scope*

This Appendix provides minimum requirements for design of AAC masonry. AAC masonry shall comply with the requirements of Chapter 1, Section A.1, and either Section A.2 or A.3.

A.1.2 *Required strength*

Required strength shall be determined in accordance with the strength design load combinations of the legally adopted building code. When the legally adopted building code does not provide load combinations, structures and members shall be designed to resist the combination of loads specified in ASCE 7-02. Members subject to compressive axial load shall be designed for the maximum design moment accompanying the axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

A.1.3 *Design strength*

AAC masonry members shall be proportioned so that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength-reduction factor, ϕ, as specified in Section A.1.5.

The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength (M_n) of the member, except that the nominal shear strength (V_n) need not exceed 2.5 times required shear strength (V_u).

For seismic design, at each story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Along each column line at a particular story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Where seismic loads are determined based on a seismic response modification factor, R, not greater than 1.5, piers and columns shall be permitted to be used to provide seismic load resistance.

A.1.4 *Strength of joints*

AAC masonry members shall be made of AAC masonry units. The tensile bond strength of AAC masonry joints shall not be taken greater than the limits of Section A.1.8.3. Head joints shall be permitted to be left unfilled between AAC masonry units laid in running bond, provided that shear capacity is calculated using the formulas of this Code corresponding to that condition.

A.1.5 *Strength-reduction factors*

A.1.5.1 *Combinations of flexure and axial load in reinforced AAC masonry* — The value of ϕ shall be taken as 0.90 for reinforced AAC masonry designed for flexure, axial load, or combinations thereof.

A.1.5.2 *Combinations of flexure and axial load in unreinforced AAC masonry* — The value of ϕ shall be taken as 0.60 for unreinforced AAC masonry designed for flexure, axial load, or combinations thereof.

A.1.5.3 *Shear* — The value of ϕ shall be taken as 0.80 for AAC masonry designed for shear.

A.1.5.4 *Anchor bolts* — For cases where the nominal strength of an anchor bolt is controlled by AAC masonry breakout, ϕ shall be taken as 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, ϕ shall be taken as 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, ϕ shall be taken as 0.65.

A.1.5.5 *Bearing* — For cases involving bearing on AAC masonry, ϕ shall be taken as 0.60.

A.1.6 *Deformation requirements*

A.1.6.1 *Drift limits* — Under loading combinations that include earthquake, masonry structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from the legally adopted building code. When the legally adopted building code does not provide allowable story drifts, structures shall be designed so the calculated story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from ASCE 7-02.

For determining drift, the calculated deflection shall be multiplied by C_d as indicated in the legally adopted building code. When the legally adopted building code does not provide C_d values, the deflection amplification factors shall be taken from ASCE 7-02.

A.1.6.2 *Deflection of unreinforced (plain) AAC masonry* — Deflection calculations for unreinforced (plain) AAC masonry members shall be based on uncracked section properties.

A.1.6.3 *Deflection of reinforced AAC masonry* — Deflection calculations for reinforced AAC masonry members shall be based on cracked section properties including the reinforcement and grout. The flexural and shear stiffness properties assumed for deflection calculations shall not exceed one-half of the gross section properties unless a cracked-section analysis is performed.

A.1.7 *Anchor bolts*

Headed and bent-bar anchor bolts shall be embedded in grout, and shall be designed in accordance with Section 3.1.6 using f'_g instead of f'_m and neglecting the contribution of AAC to the edge distance and embedment depth. Anchors embedded in AAC without grout shall be designed using nominal capacities provided by the anchor manufacturer and verified by an independent testing agency.

A.1.8 *Material properties*

A.1.8.1 *Compressive strength*

A.1.8.1.1 *Masonry compressive strength* — The specified compressive strength of AAC masonry, f'_{AAC}, shall equal or exceed 290 psi (3.45 MPa).

A.1.8.1.2 *Grout compressive strength* — The specified compressive strength of grout, f'_g, shall equal or exceed 2,000 psi (13.8 MPa) and shall not exceed 5,000 psi (34.5 MPa).

A.1.8.2 *Masonry splitting tensile strength* – The splitting tensile strength $f_{t\,AAC}$ shall be determined by Eq. A-1.

$$f_{t\,AAC} = 2.4\sqrt{f'_{AAC}} \qquad \text{(A-1)}$$

A.1.8.3 *Masonry modulus of rupture* — The modulus of rupture, f_{rAAC}, for AAC masonry elements shall be taken as two times the masonry splitting tensile strength, f_{tAAC}. If a section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 50 psi (345 kPa) at that section. If a section of AAC contains a bed joint between thin-bed mortar and AAC, the value of f_{rAAC} shall not exceed 80 psi (552 kPa) at that section.

A.1.8.4 *Masonry direct shear strength* – The direct shear strength, f_v, shall be determined by Eq. A-2.

$$f_v = 0.15 f'_{AAC} \qquad \text{(A-2)}$$

A.1.8.5 *Coefficient of friction* – The coefficient of friction between AAC and AAC shall be 0.75. The coefficient of friction between AAC and thin-bed mortar or between AAC and leveling-bed mortar shall be 1.0.

A.1.8.6 *Reinforcement strength* — Masonry design shall be based on a reinforcement strength equal to the specified yield strength of reinforcement, f_y, which shall not exceed 60,000 psi (413.7 MPa). The actual yield strength shall not exceed 1.3 times the specified yield strength. The compressive resistance of steel reinforcement shall be neglected, unless lateral reinforcement is provided in compliance with the requirements of Section 2.1.6.5.

A.1.9 *Section properties*

Member strength shall be computed using section properties based on the minimum net cross-sectional area of the member under consideration. Section properties shall be based on specified dimensions.

A.1.10 *Concentrated loads*

A.1.10.1 For computing compressive capacity for walls laid in running bond, the effective cross-sectional area shall be taken as the product of the width of the bearing area times a length not to exceed the width of the bearing area plus four times the thickness of the supporting wall, or the center-to-center distance between concentrated loads.

A.1.10.2 Bearing capacity shall be computed using a bearing area determined as follows:

(a) The direct bearing area A_1, or

(b) $A_1\sqrt{A_2 / A_1}$ but not more than $2A_1$, where A_2 is the supporting surface wider than A_1 on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as upper base sloping at 45 degrees from the horizontal and wholly contained within the support. For walls in other than running bond, area A_2 shall terminate at head joints.

A.1.10.3 Design bearing strength of AAC masonry shall equal $\phi f'_{AAC}$ multiplied by the bearing area defined in A.1.10.2 (a) or A.1.10.2 (b).

A.1.10.4 *Bearing for simply supported precast floor and roof members on AAC masonry shear walls* — The following minimum requirements shall apply so that after the consideration of tolerances, the distance from the edge of the supporting wall to the end of the precast member in the direction of the span is not less than:

For AAC floor panels	2 in. (51 mm)
For solid or hollow-core slabs	2 in. (51 mm)
For beams or stemmed members	3 in. (76 mm)

A.2 —Unreinforced (plain) AAC masonry

A.2.1 *Scope*

The requirements of Section A.2 are in addition to the requirements of Chapter 1 and Section A.1, and govern masonry design in which AAC masonry is used to resist tensile forces.

A.2.1.1 *Strength for resisting loads* — Unreinforced (plain) AAC masonry members shall be designed using the strength of masonry units, mortar, and grout in resisting design loads.

A.2.1.2 *Strength contribution from reinforcement* — Stresses in reinforcement shall not be considered effective in resisting design loads.

A.2.1.3 *Design criteria* — Unreinforced (plain) AAC masonry members shall be designed to remain uncracked.

A.2.2 *Flexural strength of unreinforced (plain) AAC masonry members*

The following assumptions shall apply when determining the flexural strength of unreinforced (plain) AAC masonry members:

(a) Strength design of members for factored flexure and axial load shall be in accordance with principles of engineering mechanics.

(b) Strain in masonry shall be directly proportional to the distance from the neutral axis.

(c) Flexural tension in masonry shall be assumed to be directly proportional to strain.

(d) Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed directly proportional to strain. Nominal compressive strength shall not exceed a stress corresponding to 0.85 f'_{AAC}.

(e) The nominal flexural tensile strength of AAC masonry shall be determined from Section A.1.8.3.

A.2.3 *Nominal axial strength of unreinforced (plain) AAC masonry members*

Nominal axial strength, P_n, shall be computed using Eq. (A-3) or Eq. (A-4).

(a) For members having an *h/r* ratio not greater than 99:

$$P_n = 0.80\left\{0.85 A_n\, f'_{AAC}\left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \qquad \text{(A-3)}$$

(b) For members having an *h/r* ratio greater than 99:

$$P_n = 0.80\left[0.85 A_n\, f'_{AAC}\left(\frac{70\,r}{h}\right)^2\right] \qquad \text{(A-4)}$$

A.2.4 *Nominal shear strength of unreinforced (plain) AAC masonry members*

The nominal shear strength of AAC masonry, V_{AAC}, shall be the least of the values computed by Sections A.3.4.1.2.1 through A.3.4.1.2.3. In evaluating nominal shear strength by Section A.3.4.1.2.3, effects of reinforcement shall be neglected. The provisions of A.3.4.1.2 shall apply to AAC shear walls laid in other than running bond.

A.2.5 *Flexural cracking*

The flexural cracking strength shall be computed in accordance with Section A.3.6.5.

A.3 — Reinforced AAC masonry

A.3.1 *Scope*

The requirements of this section are in addition to the requirements of Chapter 1 and Section A.1 and govern AAC masonry design in which reinforcement is used to resist tensile forces.

A.3.2 *Design assumptions*

The following assumptions apply to the design of reinforced AAC masonry:

(a) There is strain continuity between the reinforcement, grout, and AAC masonry such that applicable loads are resisted in a composite manner.

(b) The nominal strength of singly reinforced AAC masonry cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium.

(c) The maximum usable strain, ε_u, at the extreme AAC masonry compression fiber shall be assumed to be 0.003.

(d) Strain in reinforcement and AAC masonry shall be assumed to be directly proportional to the distance from the neutral axis.

(e) Stress in reinforcement shall be taken as E_s times steel strain but not greater than f_y.

(f) The tensile strength of AAC masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

(g) The relationship between AAC masonry compressive stress and masonry strain shall be assumed to be defined by the following: AAC masonry stress of 0.85 f'_{AAC} shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance a = 0.67 c from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis shall be measured perpendicular to that axis.

A.3.3 *Reinforcement requirements and details*

A.3.3.1 *Reinforcing bar size limitations* — Reinforcing bars used in AAC masonry shall not be larger than No. 9 (M#29). The nominal bar diameter shall not exceed one-eighth of the nominal member thickness and shall not exceed one-quarter of the least clear dimension of the cell, course, or collar joint in which it is placed. In plastic hinge zones, the area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 3 percent of the cell area. In other than plastic hinge zones, the area of reinforcing bars placed in a cell or in a course of hollow unit construction shall not exceed 4.5 percent of the cell area.

A.3.3.2 Standard hooks — The equivalent embedment length to develop standard hooks in tension, l_e, shall be determined by Eq. (A-5):

$$l_e = 13 d_b \qquad \text{(A-5)}$$

A.3.3.3 *Development*

A.3.3.3.1 *Development of tension and compression reinforcement* — The required tension or compression reinforcement shall be developed in accordance with the following provisions:

The required development length of reinforcement shall be determined by Eq. (A-6), but shall not be less than 12 in. (305 mm).

$$l_d = \frac{0.13\, d_b^{\,2} f_y \gamma}{K_{AAC}\sqrt{f'_g}} \qquad \text{(A-6)}$$

K_{AAC} shall not exceed the least of the grout cover, the clear spacing between adjacent reinforcement, nor 5 times d_b.

γ = 1.0 for No. 3 (M#10) through No. 5 (M#16) bars;

γ = 1.3 for No. 6 (M#19) through No. 7 (M#22) bars; and

γ = 1.5 for No. 8 (M#25) through No. 9 (M#29) bars.

A.3.3.3.2 *Development of shear reinforcement* — Shear reinforcement shall extend the depth of the member less cover distances.

A.3.3.3.2.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section A.3.4.1.2, shall be bent around the edge vertical reinforcing bar with a 180-degree hook. The ends of single leg or U-stirrups shall be anchored by one of the following means:

(a) A standard hook plus an effective embedment of $l_d/2$. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

(b) For No. 5 (M #16) bars and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $l_d/3$. The $l_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of the member, $d/2$, and the start of the hook (point of tangency).

(c) Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

A.3.3.3.2.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section A.3.4.1.2 shall be bent around the edge vertical reinforcing bar with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

A.3.3.4 *Splices* — Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 12 in. (305 mm) or the development length determined by Eq. (A-6), whichever is greater.

(b) A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

(c) Mechanical splices shall have the bars connected to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.

A.3.3.5 *Maximum reinforcement percentages* — The ratio of reinforcement, ρ, shall be calculated in accordance with Section 3.3.3.5 with the following exceptions:

The maximum usable strain, ε_{mu}, at the extreme masonry compression fiber shall be assumed to be 0.003 for AAC masonry.

The strength of the compression zone shall be calculated as 85 percent of f'_{AAC} multiplied by 67 percent of the area of the compression zone.

A.3.3.6 *Bundling of reinforcing bars* — Reinforcing bars shall not be bundled.

A.3.4 *Design of beams, piers, and columns*

Member design forces shall be based on an analysis that considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of beams, piers, and columns. The effects of cracking on member stiffness shall be considered.

A.3.4.1 *Nominal strength*

A.3.4.1.1 *Nominal axial and flexural strength* — The nominal axial strength, P_n, and the nominal flexural strength, M_n, of a cross section shall be determined in accordance with the design assumptions of Section A.3.2 and A.3.4.1. For any value of nominal flexural strength, the corresponding nominal axial strength calculated in accordance with Sections A.3.2 and A.3.4.1 shall be modified for the effects of slenderness. The nominal flexural strength at any section along a member shall not be less than one-fourth of the maximum nominal flexural strength at the critical section.

The nominal axial compressive strength shall not exceed Eq. (A-7) or Eq. (A-8), as appropriate.

(a) For members having an h/r ratio not greater than 99:

$$P_n = 0.80\left[0.85 f'_{AAC}\left(A_n - A_s\right) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{(A-7)}$$

(b) For members having an h/r ratio greater than 99:

$$P_n = 0.80\left[0.85 f'_{AAC}\left(A_n - A_s\right) + f_y A_s\right]\left(\frac{70r}{h}\right)^2 \quad \text{(A-8)}$$

A.3.4.1.2 *Nominal shear strength* — Nominal shear strength, V_n, shall be computed using Eq. (A-9) and either Eq. (A-10) or Eq. (A-11), as appropriate.

$$V_n = V_{AAC} + V_s \quad \text{(A-9)}$$

where V_n shall not exceed the following:

(a) Where $M_u/V_u\,d_v \le 0.25$:

$$V_n \le 6A_n\sqrt{f'_{AAC}} \quad \text{(A-10)}$$

(b) Where $M_u/V_u\,d_v \ge 1.00$

$$V_n \le 4A_n\sqrt{f'_{AAC}} \quad \text{(A-11)}$$

(c) The maximum value of V_n for $M_u/V_u\,d_v$ between 0.25 and 1.0 shall be permitted to be interpolated.

The nominal masonry shear strength shall be taken as the least of the values computed using Section A.3.4.1.2.1 through A.3.4.1.2.3. Nominal shear strength provided by reinforcement, V_s, shall include only deformed reinforcement embedded in grout for AAC shear walls.

A.3.4.1.2.1 *Nominal masonry shear strength as governed by web-shear cracking* — Nominal masonry shear strength as governed by web-shear cracking, V_{AAC}, shall be computed using Eq. (A-12a) for AAC masonry with mortared head joints, and Eq. (A-12b) for masonry with unmortared head joints:

$$V_{AAC} = 0.95\, \ell_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \quad \text{(A-12a)}$$

$$V_{AAC} = 0.66\, \ell_w\, t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, \ell_w\, t}} \quad \text{(A-12b)}$$

For AAC masonry in other than running bond, nominal masonry shear strength as governed by web-shear cracking, V_{AAC}, shall be computed using Eq. (A-12c):

$$V_{AAC} = 0.9 \sqrt{f'_{AAC}}\, A_n + 0.05 P_u \quad \text{(A-12c)}$$

A.3.4.1.2.2 *Nominal shear strength as governed by crushing of diagonal compressive strut* — For walls with $M_u/V_u\, d_v < 1.5$, nominal shear strength, V_{AAC}, as governed by crushing of a diagonal strut, shall be computed as follows:

$$V_{AAC} = 0.17\, f'_{AAC}\, t \frac{h \cdot l_w^{\,2}}{h^2 + (\frac{3}{4} l_w)^2} \quad \text{(A-13a)}$$

For walls with $M_u/V_u\, d_v$ equal to or exceeding 1.5, capacity as governed by crushing of the diagonal compressive strut need not be calculated.

A.3.4.1.2.3 *Nominal shear strength as governed by sliding shear* — At an unbonded interface, nominal shear strength as governed by sliding shear, V_{AAC}, shall be as follows:

$$V_{AAC} = \mu_{AAC} P_u \quad \text{(A-13b)}$$

At an interface where thin-bed mortar or leveling-bed mortar are present, the nominal sliding shear capacity shall be calculated by Eq. A-13b using the coefficient of friction from Section A.1.8.5.

A.3.4.1.2.4 *Nominal shear strength provided by shear reinforcement* — Nominal shear strength provided by reinforcement, V_s, shall be computed as follows:

$$V_s = \left(\frac{A_v}{s}\right) f_y d_v \quad \text{(A-14)}$$

A.3.4.1.2.5 Nominal shear strength as governed by out-of-plane loading shall be computed as follows:

$$V_{AAC} = 0.8 \sqrt{f'_{AAC}}\; bd \quad \text{(A-15)}$$

A.3.4.2 *Beams*

A.3.4.2.1 Members designed primarily to resist flexure shall comply with the requirements of Section A.3.4.2. The factored axial compressive force on a beam shall not exceed $0.05\, A_n f'_{AAC}$.

A.3.4.2.2 *Longitudinal reinforcement*

A.3.4.2.2.1 The variation in longitudinal reinforcing bars shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.

A.3.4.2.2.2 The nominal flexural strength of a beam shall not be less than 1.3 times the nominal cracking moment strength of the beam, M_{cr}. The modulus of rupture, f_{rAAC}, for this calculation shall be determined in accordance with Section A.1.8.3.

A.3.4.2.3 *Transverse reinforcement* — Transverse reinforcement shall be provided where V_u exceeds ϕV_{AAC}. The factored shear, V_u, shall include the effects of lateral load. When transverse reinforcement is required, the following provisions shall apply:

(a) Transverse reinforcement shall be a single bar with a 180-degree hook at each end.

(b) Transverse reinforcement shall be hooked around the longitudinal reinforcement.

(c) The minimum area of transverse reinforcement shall be $0.0007\, bd_v$.

(d) The first transverse bar shall not be located more than one-fourth of the beam depth, d_v, from the end of the beam.

(e) The maximum spacing shall not exceed the lesser of one-half the depth of the beam or 48 in. (1219 mm).

A.3.4.2.4 *Construction* — Beams shall be grouted solid.

A.3.4.2.5 *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.

(b) The nominal depth of a beam shall not be less than 8 in. (203 mm).

A.3.4.3 *Piers*

A.3.4.3.1 The factored axial compression force on the piers shall not exceed $0.3\,A_n f'_{AAC}$.

A.3.4.3.2 *Longitudinal reinforcement* — A pier subjected to in-plane stress reversals shall be reinforced symmetrically about the geometric center of the pier. The longitudinal reinforcement of piers shall comply with the following:

(a) One bar shall be provided in the end cells.
(b) The minimum area of longitudinal reinforcement shall be 0.0007 *bd*.
(c) Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

A.3.4.3.3 *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) The nominal thickness of a pier shall not be less than 6 in. (152 mm) and shall not exceed 16 in. (406 mm).
(b) The distance between lateral supports of a pier shall not exceed 25 times the nominal thickness of a pier except as provided for in Section A.3.4.3.3(c).
(c) When the distance between lateral supports of a pier exceeds 25 times the nominal thickness of the pier, design shall be based on the provisions of Section A.3.5.
(d) The nominal length of a pier shall not be less than three times its nominal thickness nor greater than six times its nominal thickness. The clear height of a pier shall not exceed five times its nominal length.
Exception: When the factored axial force at the location of maximum moment is less than $0.05\,f'_{AAC}A_g$, the length of a pier shall be permitted to be taken equal to the thickness of the pier.

A.3.4.4 *Columns*

A.3.4.4.1 *Longitudinal reinforcement* — Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:

(d) Maximum reinforcement area shall be determined in accordance with Section A.3.3.5, but shall not exceed $0.04\,A_n$.
(e) Minimum reinforcement area shall be $0.0025\,A_n$.
(f) Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

A.3.4.4.2 *Lateral ties* — Lateral ties shall be provided in accordance with Section 2.1.6.5.

A.3.4.4.3 *Construction* — Columns shall be solid grouted.

A.3.4.4.4 *Dimensional limits* — Dimensions shall be in accordance with the following:

(a) The nominal width of a column shall not be less than 8 in. (203 mm).
(b) The distance between lateral supports of a column shall not exceed 30 times its nominal width.
(c) The nominal depth of a column shall not be less than 8 in. (203 mm) and not greater than three times its nominal width.

A.3.5 *Wall design for out-of-plane loads*

A.3.5.1 *General* — The requirements of Section A.3.5 are for the design of walls for out-of-plane loads.

A.3.5.2 *Maximum reinforcement* — The maximum reinforcement ratio shall be determined by Section A.3.3.5.

A.3.5.3 *Moment and deflection calculations* — Moment and deflection calculations in Section A.3.5.4 are based on simple support conditions top and bottom. For other support and fixity conditions, moments, and deflections shall be calculated using established principles of mechanics.

A.3.5.4 *Walls with factored axial stress of $0.2\,f'_{AAC}$ or less* — The procedures set forth in this section shall be used when the factored axial load stress at the location of maximum moment satisfies the requirement computed by Eq. (A-16).

$$\left(\frac{P_u}{A_g}\right) \le 0.2\,f'_{AAC} \tag{A-16}$$

Factored moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be computed using Eq. (A-17).

$$M_u = \frac{w_u h^2}{8} + P_{uf}\,\frac{e_u}{2} + P_u \delta_u \tag{A-17}$$

Where:

$$P_u = P_{uw} + P_{uf} \tag{A-18}$$

The design strength for out-of-plane wall loading shall be in accordance with Eq. (A-19).

$$M_u \le \phi M_n \tag{A-19}$$

Where:

$$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right) \tag{A-20}$$

$$a = \frac{\left(P_u + A_s f_y\right)}{0.85\,f'_{AAC}\,b} \tag{A-21}$$

The nominal shear strength for out-of-plane loads shall be determined by Section A.3.4.1.2.5.

A.3.5.5 *Walls with factored axial stress greater than $0.2\,f'_{AAC}$ or slenderness ratios greater than 30* — Such walls shall be designed in accordance with the provisions of Section A.3.5.4 and shall have a minimum nominal thickness of 6 in. (152 mm).

The nominal shear strength shall be determined by Section A.3.4.1.2.5.

A.3.5.6 *Deflection design* — The horizontal midheight deflection, δ_s, under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \leq 0.007\, h \tag{A-22}$$

P-delta effects shall be included in deflection calculation. The midheight deflection shall be computed using either Eq. (A-23) or Eq. (A-24), as applicable.

(a) Where $M_{ser} < M_{cr}$

$$\delta_s = \frac{5 M_{ser} h^2}{48 E_{AAC} I_g} \tag{A-23}$$

(b) Where $M_{cr} < M_{ser} < M_n$

$$\delta_s = \frac{5 M_{cr} h^2}{48 E_{AAC} I_g} + \frac{5\left(M_{ser} - M_{cr}\right) h^2}{48 E_{AAC} I_{cr}} \tag{A-24}$$

The cracking moment strength of the wall shall be computed using Eq. (A-25), where f_{rAAC} is given by Section A.1.8.3:

$$M_{cr} = S_n \left(f_{rAAC} + \frac{P}{A_n} \right) \tag{A-25}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 60 psi (414 kPa).

A.3.6 *Wall design for in-plane loads*

A.3.6.1 *Scope* — The requirements of Section A.3.6 are for the design of walls to resist in-plane loads.

A.3.6.2 *Reinforcement* — Reinforcement shall be in accordance with the following:

(a) The amount of vertical reinforcement shall not be less than one-half the horizontal reinforcement.

(b) The maximum reinforcement ratio shall be determined in accordance with Section A.3.3.5.

A.3.6.3 *Flexural and axial strength* — The nominal flexural and axial strength shall be determined in accordance with Section A.3.4.1.1.

A.3.6.4 *Shear strength* — The nominal shear strength shall be computed in accordance with Section A.3.4.1.2.

A.3.6.5 *Flexural cracking strength* — The flexural cracking strength shall be computed in accordance with Eq. (A-26), where f_{rAAC} is given by Section A.1.8.3:

$$V_{cr} = \frac{S_n}{h} \left(f_{rAAC} + \frac{P}{A_n} \right) \tag{A-26}$$

If the section of AAC masonry contains a horizontal leveling bed, the value of f_{rAAC} shall not exceed 60 psi (414 kPa).

A.3.6.6 The maximum reinforcement requirements of Section A.3.3.5 shall not apply if a shear wall is designed to satisfy the requirements of Sections A.3.6.7 through A.3.6.10.

A.3.6.7 The need for special boundary elements at the edges of shear walls shall be evaluated in accordance with Section A.3.6.8 or A.3.6.9. The requirements of Section A.3.6.10 shall also be satisfied.

A.3.6.8 This Section applies to walls bending in single curvature in which the flexural limit state response is governed by yielding at the base of the wall. Walls not satisfying those requirements shall be designed in accordance with Section A.3.6.9.

(a) Special boundary elements shall be provided over portions of compression zones where:

$$c \geq \frac{l_w}{600 \left(C_d \delta_{ne} / h_w \right)}$$

and c is calculated for the P_u given by ASCE 7-02 Load Combination 5 (1.2D + 1.0E + L + 0.2S) or the corresponding strength design load combination of the legally adopted building code, and the corresponding nominal moment strength, M_n, at the base critical section. The load factor on L in Load Combination 5 is reducible to 0.5, as per exceptions to Section 2.3.2 of ASCE 7-02.

(b) Where special boundary elements are required by Section A.3.6.8 (a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w or $M_u/4V_u$.

A.3.6.9 Shear walls not designed to the provisions of Section A.3.6.8 shall have special boundary elements at boundaries and edges around openings in shear walls where the maximum extreme fiber compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2 f'_{AAC}$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15 f'_{AAC}$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Section 1.9.4.2.3 shall be used.

A.3.6.10 Where special boundary elements are required by Section A.3.6.8 or A.3.6.9, (a) through (d) shall be satisfied and tests shall be performed to verify the strain capacity of the element:

(a) The special boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of (c - 0.1l_w) and c/2.

(b) In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 12 in. (305 mm) into the web.

(c) Special boundary element transverse reinforcement at the wall base shall extend into the support at least the development length of the largest longitudinal reinforcement in the boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 12 in. (305 mm) into the footing or mat.

(d) Horizontal shear reinforcement in the wall web shall be anchored to develop the specified yield strength, f_y, within the confined core of the boundary element.