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DIVISION: 04—MASONRY
Section: 04220—Concrete Masonry Units

REPORT HOLDER:

AUTOCLAVED AERATED CONCRETE PRODUCT ASSOCIATION (AACPA)
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EVALUATION SUBJECT:

AUTOCLAVED AERATED CONCRETE (AAC) BLOCK MASONRY UNITS

ADDITIONAL LISTEE:

E-CRETE, LLC
22580 WEST MARICOPA HIGHWAY
CASA GRANDE, ARIZONA 85222

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2003 *International Building Code*® (IBC)
- 1997 *Uniform Building Code*™ (UBC)

Properties evaluated:

- Structural
- Thermal resistance
- Noncombustibility
- Weather exposure

2.0 USES

The autoclaved aerated concrete (AAC) block units are used to construct interior and exterior masonry walls, columns, and beams. The walls shall be reinforced or unreinforced. Uses include load-bearing, nonload-bearing, and shear wall applications, along with thermal resistance.

3.0 DESCRIPTION

3.1 General:

AAC is a lightweight, noncombustible siliceous material manufactured from a mixture of sand, fly ash, mine tailings, slag or other sources of silica; portland cement; quick lime; anhydrite; water; and aluminum powder or paste. These raw materials are mixed together and cast into large steel molds where a chemical reaction takes place. Hydrogen gas is generated in the wet slurry mixture, causing it to expand and form independent air cells.

Upon setting and before hardening, the AAC material is cut into precast block units for masonry wall construction. The AAC precision blocks are steam-cured under pressure in an autoclave, where the material is transformed into its finished state.

3.2 Materials:

3.2.1 Blocks: AAC blocks are manufactured in classes AAC2, AAC4, and AAC6, with various compressive strengths and corresponding densities complying with ASTM C 1386. The AAC blocks are also provided in class AAC3. The AAC3 class has a minimum compressive strength of 435 psi (3 MPa) and a nominal dry bulk density of 31 lb/ft^3 (500 kg/m^3), with density ranging from 28 lb/ft^3 (450 kg/m^3) to 34 lb/ft^3 (550 kg/m^3).

3.2.2 Block Assemblages: AAC precast block units are manufactured in accordance with ASTM C 1386 and are assembled using AAC thin- or thick-bed mortar. The assemblies are used for exterior and interior bearing and nonbearing walls and partitions, columns, and beams. Walls, columns, and beams are constructed from blocks as unreinforced and reinforced, as dictated by the structural design. Figure 1 provides illustrations.

3.2.3 AAC Mortar: AAC $^1/_{16}$-inch-thick (1.6 mm) thin-bed and $^3/_8$-inch-thick (9.5 mm) thick-bed mortars are mixtures composed of modified polymer, white or gray portland cement, and adhesives with inorganic fillers. The mortars are prebagged in a dry, ready-mixed state. Mixing instructions are provided for the addition of water and the appropriate mixing sequence. AAC thin-bed and thick-bed mortars are glue mortars complying with ASTM C 1386 requirements for physical compatibility and strength. AAC mortars are used with AAC products of all densities and strengths. The mortars have an unlimited shelf life when stored in unopened containers.

3.3 Thermal Characteristics:

Thermal conductivity and resistance properties for AAC block units are indicated in Table 1.

3.4 Surface Treatment:

Exterior coverings complying with the UBC or IBC shall be applied on the exterior surface of exterior walls per ASTM C 1555-03a, Section 9. The interior surfaces shall be coated with conventional plaster or other approved interior wall finishes. For basement wall applications, precautions shall be taken to allow for the drying of the wall prior to backfilling. Precaution is manifested either by applying impermeable protection on the outside, below ground level, in which case heat-drying takes place towards the inside; or by applying a no-capillary air-permeable cover on the outside, below ground level (e.g., mineral wool mat). In the latter case, the drying takes place through both sides of the basement wall.

3.5 Fasteners:

Except as noted in Section 4.1.1.7, connections, using fasteners approved by the AAC manufacturer, are subject to approval by the code official for each project.

3.6 Grout:

Grout shall comply with ASTM C 476.

4.0 DESIGN AND INSTALLATION

4.1 Strength Design of AAC Masonry Structures:

4.1.1 General:

4.1.1.1 Scope: This section provides minimum requirements for strength design of AAC masonry structures. AAC masonry structures shall comply with the requirements of Section 4.1.1 and either Section 4.1.2 or Section 4.1.3 of this evaluation report. Notations not defined in this evaluation report are found in Section 2102.1 of the IBC, Section 1.5 of ACI 530-02 or Section 2101.4 of the UBC.

4.1.1.2 Required Strength: Required strength shall be determined in accordance with the strength design load combinations of Section 1612.2 of the UBC or Section 1605.2 of the IBC, as applicable. Masonry members subject to compressive axial load shall be designed for the maximum factored design moment accompanying the factored axial load. The factored moment, M_u, shall include the moment induced by relative lateral displacement.

4.1.1.3 Design Strength: AAC masonry members shall be proportioned such that the design strength exceeds the required strength. Design strength is the nominal strength multiplied by the strength reduction factor, ϕ, as specified in Section 4.1.1.5.

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 the required shear strength (V_u).

4.1.1.3.1 Seismic Design Provisions: The seismic design coefficients and factors are described in Table 2. 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.

4.1.1.3.2 Plain AAC Masonry Shear Walls:

4.1.1.3.2.1 Ordinary Plain (Unreinforced) AAC Masonry Shear Walls: The design of ordinary plain (unreinforced) AAC masonry shear walls shall comply with Section 4.1.3.

4.1.1.3.2.2 Detailed Plain (Unreinforced) AAC Masonry Shear Walls: The design of detailed plain (unreinforced) AAC masonry shear walls shall comply with Sections 4.1.1.3.3 and 4.1.3.

4.1.1.3.3 Minimum Reinforcement Requirements for Members of the Lateral-force-resisting System in AAC Masonry: Vertical reinforcement of at least 0.2 in^2 (129 mm^2) shall be provided within 24 inches (610 mm) of each side of openings, within 8 inches (203 mm) of movement joints, and within 24 inches (610 mm) of the ends of walls. Reinforcement adjacent to openings need not be provided for openings smaller than 16 inches (406 mm), unless the minimum reinforcement requirements are interrupted by such openings. Horizontal reinforcement shall be provided at the bottom and top of wall openings and shall extend not less than 24 inches (610 mm) nor less than 40 bar diameters past the opening.

4.1.1.3.4 Ordinary Reinforced AAC Masonry Shear Walls: Design of ordinary reinforced AAC masonry shear walls shall comply with the requirements of Section 4.1.1.3.3 and the requirements of Section 4.1.2.

4.1.1.3.5 Seismic Design Category A (IBC) or Seismic Zone 0 (UBC): AAC masonry structures in Seismic Design Category A (IBC) or Seismic Zone 0 (UBC) shall comply with the requirements of Section 4.1.2 or 4.1.3.

4.1.1.3.5.1 Drift Limits: The calculated story drift of masonry structures due to the combination of design seismic forces and gravity loads shall not exceed 0.007 times the story height.

4.1.1.3.5.2 Anchorage of AAC Masonry Walls: AAC masonry walls shall be anchored to the roof members and floor members that provide lateral support for the wall. The anchorage shall directly connect the walls to the roof and floor construction.

4.1.1.3.6 Seismic Design Category B (IBC) or Seismic Zone 1 (UBC): Structures in Seismic Design Category B (IBC) or Seismic Zone 1 (UBC) shall comply with the requirements of Section 4.1.1.3.5 and the additional requirements of Section 4.1.1.3.6.

4.1.1.3.6.1 Design of Elements That Are Part of the Lateral-force-resisting System: The lateral-force-resisting system shall be designed to comply with the requirements in Sections 4.1.2 and 4.1.3. Masonry shear walls shall comply with the requirements in Section 4.1.1.3.2.1 (ordinary plain [unreinforced] masonry shear walls), 4.1.1.3.2.2 (detailed plain [unreinforced] masonry shear walls), or Section 4.1.1.3.3 (ordinary reinforced masonry shear walls).

4.1.1.3.6.2 Anchorage of Floor and Roof Diaphragms in AAC Masonry Structures: Floor and roof diaphragms in AAC masonry structures shall be surrounded by a continuous grouted bond beam reinforced with at least two longitudinal reinforcing bars, having a total cross-sectional area of at least 0.4 inch^2 (260 mm^2).

4.1.1.3.7 Seismic Design Category C (IBC) or Seismic Zone 2A or 2B (UBC): Structures in Seismic Design Category C (IBC) or Seismic Zone 2A or 2B (UBC) shall comply with the requirements of Section 4.1.1.3.6 and the additional requirements of Section 4.1.1.3.7.

4.1.1.3.7.1 Design of Elements That Are Not Part of the Lateral-force-resisting System: Load-bearing frames or columns that are not part of the lateral-force-resisting system shall be analyzed as to their effect on the response of the system. Such frames or columns shall be adequate for vertical load carrying capacity and induced moment due to the design story drift.

AAC masonry partition walls, AAC masonry screen walls and other AAC masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

4.1.1.3.7.2 Design of Elements That Are Part of the Lateral-force-resisting System: Design of masonry columns and shear walls shall comply with the requirements of Sections 4.1.1.3.7.2.1 and 4.1.1.3.7.2.2. Design of ordinary reinforced AAC masonry structures shall comply with the requirements of Section 4.1.1.3.7.2.3.

4.1.1.3.7.2.1 Connections to AAC Masonry Columns: Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of Section 4.1.1.3.7.3.2.1. Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two No. 4 (M No. 13) lateral ties provided in the top 5 inches (127 mm) of the column.

4.1.1.3.7.2.2 AAC Masonry Shear Walls: Masonry shear walls shall comply with the requirements for ordinary reinforced AAC masonry shear walls in Section 4.1.1.3.4 of this report.

4.1.1.3.7.2.3 Anchorage of Floor and Roof Diaphragms in AAC Masonry Structures: Lateral load between floor and roof diaphragms and AAC masonry shear walls shall be transferred through connectors embedded in grout in accordance with Section 4.1.1.3.7.2.3.1. Connectors shall be designed to transfer horizontal design forces acting either parallel or perpendicular to the wall but not less than 200 pounds per lineal foot (2,919 N/m) of wall. The maximum connector spacing shall be 4 feet (1,219 mm) unless the wall is designed to span horizontally between connectors.

4.1.1.3.7.2.3.1 Load Transfer at Horizontal Connections: Walls, columns, and pilasters shall be designed to resist all loads, moments, and shears applied at intersections with horizontal members.

Effect of lateral deflection and translation of members providing lateral support shall be considered.

Devices used for transferring lateral support from members that intersect walls, columns, or pilasters shall be designed to resist the load involved. For columns, a load of not less than 1,000 pounds (448 N) shall be used.

4.1.1.3.8 Seismic Design Category D (IBC) or Seismic Zone 3 (UBC): Structures in Seismic Design Category D (IBC) or Seismic Zone 3 (UBC) shall comply with the requirements of Section 4.1.1.3.8.

4.1.1.3.8.1 Design Requirements: Masonry elements shall be designed in accordance with the requirements of Section 4.1.2 or 4.1.3.

4.1.1.3.8.2 Minimum Reinforcement for Masonry Columns: Lateral ties in masonry columns shall be spaced not more than 8 inches (203 mm) on center and shall be at least $^{3}/_{8}$ inch (9.5 mm) in diameter. Lateral ties shall be embedded in grout.

4.1.1.3.8.3 Material Requirements: Neither Type N mortar nor masonry cement shall be used as part of the lateral-force-resisting system.

4.1.1.3.8.4 Lateral Tie Anchorage: Standard hooks for lateral tie anchorage shall be either a 135-degree standard hook or a 180-degree standard hook.

4.1.1.3.9 Seismic Design Categories E and F (IBC) or Seismic Zone 4 (UBC): Structures in Seismic Design Categories E and F (IBC) or Seismic Zone 4 (UBC) shall comply with the requirements of Section 4.1.1.3.8 and with the additional requirements of Section 4.1.1.3.9.

4.1.1.3.9.1 Minimum Reinforcement for Stack Bond Elements That Are Not Part of the Lateral-force-resisting System: Stack bond masonry that is not part of the lateral-force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0015 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 24 inches (610 mm). These elements shall be solidly grouted and shall be constructed of hollow open end units or two wythes of solid units.

4.1.1.3.9.2 Minimum Reinforcement for Stack Bond Elements That Are Part of the Lateral-force-resisting System: Stack bond masonry that is part of the lateral-force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0025 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 16 inches (406 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

4.1.1.4 Strength of Joints: AAC masonry members shall be made of AAC masonry units and AAC mortar. The tensile bond strength of AAC joints shall not be taken greater than the limits indicated in Section 4.1.1.8.3. 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 equations of this report corresponding to that condition.

4.1.1.5 Strength Reduction Factors:

4.1.1.5.1 Combinations of Flexure and Axial Load in Reinforced AAC Masonry: The value of ϕ shall be 0.90 for reinforced AAC masonry subjected to flexure, axial load, or combinations thereof.

4.1.1.5.2 Combinations of Flexure and Axial Loads in Unreinforced AAC Masonry: The value of ϕ shall be 0.60 for unreinforced AAC masonry subjected to flexure, axial load, or combinations thereof.

4.1.1.5.3 Shear: The value of ϕ shall be 0.80 for AAC masonry designed for shear.

4.1.1.5.4 Anchor Bolts: For cases where the nominal strength of a cast-in-place anchor bolt is controlled by AAC or grout breakout, ϕ shall be 0.50. For cases where the nominal strength of an anchor bolt is controlled by anchor bolt steel, the value of ϕ shall be 0.90. For cases where the nominal strength of an anchor bolt is controlled by anchor pullout, the value of ϕ shall be 0.65.

4.1.1.5.5 Development and Splices of Reinforcement: For development and splices of reinforcement, the value of ϕ shall be 0.80.

4.1.1.5.6 Bearing: For cases involving bearing on AAC masonry, the value of ϕ shall be 0.60.

4.1.1.6 Deformation Requirements:

4.1.1.6.1 Drift Limits: Under loading combinations that include earthquake, masonry structures shall be designed so that the calculated drift, Δ, does not exceed the allowable story drift, Δ_A, obtained from Section 1617.3 of the IBC or Section 1630.10 of the UBC.

4.1.1.6.2 Deflection of Unreinforced (Plain) AAC Masonry: Deflection calculations for unreinforced (plain) AAC masonry shall be based on uncracked section properties.

4.1.1.6.3 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.

4.1.1.7 Anchor Bolts: Headed and bent-bar anchor bolts shall be embedded in grout, and shall be designed in accordance with Section 2108.6 of the IBC or Section

2108.1.5 of the UBC using f'_g instead of f'_m and neglecting the contribution of AAC to the anchor bolt's edge distance and embedment depth. For anchors embedded in AAC in mortar or AAC block units without grout, Section 3.5 applies.

4.1.1.8 Material Properties:

4.1.1.8.1 Compressive Strength:

4.1.1.8.1.1 Masonry Compressive Strength: The specified compressive strength of AAC masonry, f'_{AAC}, shall be determined in accordance with ASTM C 1386 and shall equal or exceed 290 psi (2.0 MPa).

4.1.1.8.1.2 Grout Compressive Strength: The specified compressive strength of grout, f'_g, shall be from 2,000 psi (13.8 MPa) to 5,000 psi (34.5 MPa).

4.1.1.8.2 Masonry Flexural Tensile Strength: The splitting tensile strength, $f_{t\,AAC}$, shall be determined by Equation 4-01.

$$f_{tAAC} = 2.4\sqrt{f'_{AAC}} \text{ (psi/MPa)} \qquad (4\text{-}01)$$

4.1.1.8.3 Masonry Modulus of Rupture: The modulus of rupture, f_{rAAC}, for AAC masonry block 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 masonry contains a bed joint with AAC thin-bed mortar and AAC units, the value of f_{rAAC} shall not exceed 80 psi (552 kPa) at that section.

4.1.1.8.4 Masonry Direct Shear Strength: The direct shear strength, f_v, shall be determined by Equation 4-02.

$$f_v = 0.15\, f'_{AAC} \text{ (psi/MPa)} \qquad (4\text{-}02)$$

4.1.1.8.5 Coefficient of Friction: The dry coefficient of friction, μ, between AAC masonry units shall be a maximum of 0.75. The coefficient of friction, μ, between AAC masonry units and leveling-bed mortar shall be a maximum of 1.0.

4.1.1.8.6 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 2.1.4.6 of ACI 530-02 (IBC) or Section 2108.2.3.2.3 of the UBC.

4.1.1.8.7 Modulus of Elasticity: Modulus of elasticity, E_{AAC}, of AAC masonry shall be:

$$E_{AAC} = 6{,}500\,(f'_{AAC})^{0.6}\text{, psi (Pa)} \qquad (4\text{-}03)$$

4.1.1.8.8 Modulus of Rigidity: The modulus or rigidity, E_V, of AAC masonry shall be $E_v = 0.4\, E_{AAC}$, psi (Pa). (4-04)

4.1.1.9 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.

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

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

(a) The direct bearing area, A_1, or

$$A_1\sqrt{A_2 / A_1} \qquad (4\text{-}05)$$

(b) 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.

Design bearing strength of AAC masonry shall equal ϕf_{AAC} multiplied by the bearing area defined in (a) or (b), above.

4.1.1.10.1 Bearing for Simply Supported Precast Concrete Floor and Roof Members on AAC 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 concrete member in the direction of the span is not less than:

For AAC floor panels: 2 inches (51 mm)

For solid or hollow-core concrete slabs: 2 inches (51 mm)

For beams or stemmed concrete members: 3 inches (76 mm)

4.1.2 Reinforced AAC Masonry:

4.1.2.1 Scope: The requirements of this section govern AAC masonry design in which reinforcement is used to resist tensile forces.

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

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

(b) The nominal strength of singly reinforced 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 masonry compression fiber shall be assumed to be 0.003.

(d) Strain in reinforcement and 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 no greater than f_y.

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

(g) The relationship between masonry compressive stress and masonry strain shall be assumed to be defined by the following: 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 $a = 0.67c$ 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.

4.1.2.3 Reinforcement Requirements and Details:

4.1.2.3.1 Reinforcing Bar Size Limitations: Reinforcing bars used in AAC masonry shall not be larger than No. 9 (metric No. 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.

4.1.2.3.2 Standard Hooks: The equivalent embedment length to develop standard hooks in tension, l_e, shall be determined by Equation 4-06:

$$l_e = 13d_b \tag{4-06}$$

where:

d_b = diameter of bar.

4.1.2.3.3 Development: 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 Equation 4-07, but shall not be less than 12 inches (305 mm).

$$l_d = \frac{l_{de}}{\phi} \tag{4-07}$$

where:

$$l_{de} = \frac{0.13 d_b^2 f_y \gamma}{K_{AAC}\sqrt{f_g'}} \tag{4-08}$$

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

γ = 1.0 for No. 3 (metric No. 10) through No. 5 (metric No. 16) bars.

γ = 1.4 for No. 6 (metric No. 19) through No. 7 (metric No. 22) bars.

γ = 1.5 for No. 8 (metric No. 29) through No. 9 (metric No. 29) bars.

4.1.2.3.3.1 Development of Shear Reinforcement: Shear reinforcement shall extend the depth of the member less cover distances.

Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 4.1.2.4.1.2, 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 (metric No. 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.

At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 4.1.2.4.1.2 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.

4.1.2.3.4 Splices: Reinforcement splices shall comply with one of the following:

(a) The minimum length of lap for bars shall be 12 inches (305 mm) or the length determined by Eq. (4-09), whichever is greater.

$$l_d = \frac{l_{de}}{\phi} \tag{4-09}$$

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

4.1.2.3.5 Maximum Area of Flexural Tensile Reinforcement: For masonry members where $M_u/(V_u\, d_v) >$ 1.00, the cross-sectional area of flexural tensile reinforcement shall not exceed the area required to maintain axial equilibrium under the following conditions:

(a) A strain gradient shall be assumed, corresponding to a strain in the extreme tension reinforcement equal to 1.5 times yield strain and a maximum strain in the AAC masonry of 0.003.

(b) The design assumptions of Section 4.1.2.2 shall apply.

(c) The stress in the tension reinforcement shall be taken as the product of the modulus of elasticity of the steel and strain in the reinforcement, and need not be taken as greater than f_y.

(d) Axial forces shall be taken from the loading combination $D + 0.75L + 0.525Q_E$.

where:

Q_E = Effect of horizontal seismic forces.

(e) The effect of compression reinforcement, with or without lateral restraining reinforcement, shall be permitted to be included for the purposes of calculating maximum tensile reinforcement.

For masonry members where $M_u/V_u\, d_v \leq 1.00$ and when designed using an R less than or equal to 1.5, there is no upper limit to maximum flexural tensile reinforcement. For masonry members where $M_u/V_u\, d_v \leq 1.00$ and when designed using an R greater than 1.5, the provisions of Section 4.1.2.3.5 shall apply.

4.1.2.3.6 Bundling of Reinforcing Bars: Reinforcing bars shall not be bundled.

4.1.2.4 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 all beams, piers, and columns. The effects of cracking on member stiffness shall be considered.

4.1.2.4.1 Nominal Strength:

4.1.2.4.1.1 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 Sections 4.1.2.2 and 4.1.2.4.1. For any value of nominal flexural strength, the corresponding nominal axial strength, calculated in accordance with Sections 4.1.2.2 and 4.1.2.4.1, 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. (4-10) or Eq. (4-11), as appropriate.

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

$$P_n = 0.80\,[0.85\,f'_{AAC}\,(A_n - A_s) + f_y\,A_s]\left(1-\left(\frac{h}{140r}\right)^2\right) \quad (4\text{-}10)$$

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

$$P_n = 0.80\,[0.85\,f'_{AAC}\,(A_n - A_s) + f_y\,A_s]\left(\frac{70r}{h}\right)^2 \quad (4\text{-}11)$$

where:

A_n = Net cross-sectional area of AAC, square inches (mm^2).

h = Effective height of a column, pilaster or wall, inches (mm).

P_n = Nominal axial strength in AAC, pounds (N).

r = Radius of gyration, inches (mm).

4.1.2.4.1.2 Nominal Shear Strength: Nominal shear strength, V_n, shall be computed using Eq. (4-12) and either Eq. (4-13) or Eq. (4-14), as appropriate.

$$V_n = V_{AAC} + V_s \quad (4\text{-}12)$$

where V_n shall not exceed the following:

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

$$V_n \leq 6\,A_n\,\sqrt{f'_{AAC}} \quad (4\text{-}13)$$

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

$$V_n \leq 4\,A_n\,\sqrt{f'_{AAC}} \quad (4\text{-}14)$$

(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 4.1.2.4.1.2.1 through 4.1.2.4.1.2.3. Nominal shear strength provided by reinforcement, V_s, shall include only deformed reinforcement embedded in grout for AAC shear walls.

4.1.2.4.1.2.1 Nominal In-plane Masonry Shear Strength as Governed by Web-shear Cracking: Nominal masonry shear strength as governed by web-shear cracking, V_m, shall be computed using Eq. (4-15a) for AAC masonry with mortared head joints, and Eq. (4-15b) for masonry with unmortared head joints:

$$V_{AAC} = 0.95\,\ell_w\,t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,\ell_w t}} \quad (4\text{-}15a)$$

$$V_{AAC} = 0.66\,\ell_w\,t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,\ell_w t}} \quad (4\text{-}15b)$$

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. (4-15c):

$$V_{AAC} = 0.9\,\sqrt{f'_{AAC}}\,A_n + 0.05P_u \quad (4\text{-}15c)$$

4.1.2.4.1.2.2 Nominal In-plane 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:

$$V_{AAC} = 0.17\,f'_{AAC}\,t\,\frac{h \cdot l_w^2}{h^2 + (\frac{3}{4} l_w)^2} \quad (4\text{-}16a)$$

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.

4.1.2.4.1.2.3 Nominal In-plane 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:

$$V_{AAC} = \mu P_u \quad (4\text{-}16b)$$

At an interface where AAC thin-bed mortar or leveling bed mortar are present, the nominal sliding shear capacity shall be calculated by Eq. (4-16b) using coefficient of friction, μ, from Section 4.1.1.8.5.

4.1.2.4.1.2.4 Nominal Shear Strength Provided by Shear Reinforcement: Nominal shear strength provided by reinforcement, V_s, shall be computed as follows:

$$V_s = \frac{Av}{s}\,f_y d_v \quad (4\text{-}17)$$

4.1.2.4.1.2.5 Nominal Shear Strength as Governed by Out-of-plane Loading: Nominal shear strength as governed by out-of-plane loading shall be computed as follows:

$$V_{AAC} = 0.8\,\sqrt{f'_{AAC}}\,bd \quad (4\text{-}18)$$

4.1.2.4.2 Beams: AAC masonry members designed primarily to resist flexure shall comply with the requirements of Section 4.1.2.4.2. The factored axial compressive force on a beam shall not exceed 0.05 $A_n\,f'_{AAC}$.

4.1.2.4.2.1 Longitudinal Reinforcement: 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.

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_r, for this calculation shall be determined in accordance with Section 4.1.1.8.3.

4.1.2.4.2.2 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 one half the depth of the beam nor 48 inches (1,219 mm).

4.1.2.4.2.3 Construction: Beams shall be grouted solid.

4.1.2.4.2.4 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 inches (203 mm).

4.1.2.4.3 Piers: The factored axial compression force on the piers shall not exceed 0.3 $A_n f'_{AAC}$.

4.1.2.4.3.1 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.

4.1.2.4.3.2 Dimensional Limits: Dimensions shall be in accordance with the following:

(a) The nominal thickness of a pier shall not be less than 6 inches (152 mm) and shall not exceed 16 inches (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 4.1.2.4.3.2(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 4.1.2.5.

(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 equal to the thickness of the pier.

4.1.2.4.4 Columns:

4.1.2.4.4.1 Longitudinal Reinforcement: Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column, and shall comply with the following:

(a) Maximum reinforcement area shall be determined in accordance with Section 4.1.2.3.5, but shall not exceed 0.04 A_n.

(b) Minimum reinforcement area shall be 0.0025 A_n.

(c) Longitudinal reinforcement shall be uniformly distributed throughout the depth of the element.

4.1.2.4.4.2 Lateral Ties: Lateral ties shall be provided in accordance with Section 2.1.6.5 of ACI 530-02 (IBC) or Section 2108.2.3.2.3 of the UBC.

4.1.2.4.4.3 Construction: Columns shall be solid grouted.

4.1.2.4.4.4 Dimensional Limits: Dimensions shall be in accordance with the following:

(a) The nominal width of a column shall not be less than 8 inches (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 inches (203 mm) and not greater than three times the column's nominal width.

4.1.2.5 Wall Design for Out-of-plane Loads:

4.1.2.5.1 General: The requirements of Section 4.1.2.5 are for the design of walls for out-of-plane loads.

4.1.2.5.2 Maximum Reinforcement: The maximum reinforcement ratio shall be determined in accordance with Section 4.1.2.3.5.

4.1.2.5.3 Moment and Deflection Calculations: Moment and deflection calculations in Section 4.1.2.5.4 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.

4.1.2.5.4 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. (4-19).

$$\frac{P_u}{A_g} \leq 0.2\, f'_{AAC} \tag{4-19}$$

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. (4-20).

$$M_u = \frac{w_u h^2}{8} + P_{uf}\,\frac{e_u}{2} + P_u \delta_u \tag{4-20}$$

where:

$$P_u = P_{uw} + P_{uf} \tag{4-21}$$

The design strength for out-of-plane wall loading shall be in accordance with Eq. (4-22).

$$M_u \leq \phi M_n \tag{4-22}$$

where:

$$M_n = (A_s f_y + P_u)\left(d - \frac{a}{2}\right) \tag{4-23}$$

$$a = \frac{(P_u + A_s f_y)}{0.85 f'_{AAC} b} \tag{4-24}$$

The nominal shear strength for out-of-plane loads shall be determined by Section 4.1.2.4.1.2.5.

4.1.2.5.5 Walls with Factored Axial Stress Greater than 0.2f'_{AAC} or Slenderness Ratios Greater than 30: Such walls shall be designed in accordance with the provisions of Section 4.1.2.5.4 and shall have a minimum nominal thickness of 6 inches (152 mm).

The nominal shear strength shall be determined by Section 4.1.2.4.1.2.

4.1.2.5.6 Deflection Design: The horizontal midheight deflection, δ_s, under service lateral and service axial loads (without load factors) shall be limited by the relation:

$$\delta_s \leq 0.007h \tag{4-25}$$

P-delta effects shall be included in deflection calculations. The midheight deflection shall be computed using either Eq. (4-26) or Eq. (4-27), as applicable.

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

$$\delta_s = \frac{5 M_{ser} h^2}{48 E_{AAC} I_g} \tag{4-26}$$

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

$$\delta_s = \frac{5M_{cr}h^2}{48E_{AAC}I_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_{AAC}I_{cr}} \quad (4\text{-}27)$$

The cracking moment strength of the wall shall be computed using Eq. (4-28), where f_{rAAC} is given by Section 4.1.1.8.3:

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right) \quad (4\text{-}28)$$

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

4.1.2.6 Wall Design for In-plane Loads:

4.1.2.6.1 Scope: The requirements of Section 4.1.2.6 are for the design of walls to resist in-plane loads.

4.1.2.6.2 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 4.1.2.3.5.

4.1.2.6.3 Flexural and Axial Strength: The nominal flexural and axial strength shall be determined in accordance with Section 4.1.2.4.1.1.

4.1.2.6.4 Shear Strength: The nominal shear strength shall be computed in accordance with Section 4.1.2.4.1.2.

4.1.2.6.5 Flexural Cracking Strength: The flexural cracking strength shall be computed in accordance with Eq. (4-29), where f_{rAAC} is given by Section 4.1.1.8.3:

$$V_{cr} = \frac{S_n}{h} \cdot \left(f_{rAAC} + \frac{P}{A_n}\right) \quad (4\text{-}29)$$

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

4.1.3 Unreinforced (Plain) AAC Masonry:

4.1.3.1 Scope: The requirements of this section govern masonry design in which AAC masonry is used to resist tensile forces.

4.1.3.1.1 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.

4.1.3.1.2 Strength Contribution from Reinforcement: Stresses in reinforcement shall not be considered effective in resisting design loads.

4.1.3.1.3 Design Criteria: Unreinforced (plain) AAC masonry members shall be designed to remain uncracked.

4.1.3.2 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 4.1.1.8.2.

4.1.3.3 Nominal Axial Strength of Unreinforced (Plain) AAC Masonry Members: Nominal axial strength, P_n, shall be computed using Eq. (4-30) or Eq. (4-31).

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

$$P_n = 0.80\left(0.85A_n f'_{AAC}\left[1 - \left(\frac{h}{140r}\right)^2\right]\right) \quad (4\text{-}30)$$

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

$$P_n = 0.80\left(0.85A_n f'_{AAC}\left(\frac{70r}{h}\right)^2\right) \quad (4\text{-}31)$$

4.1.3.4 Nominal Shear Strength of Unreinforced (Plain) Masonry Members: The nominal shear strength of AAC masonry, V_{AAC}, shall be the least of the values computed by Sections 4.1.2.4.1.2.1 through 4.1.2.4.1.2.3. In evaluating nominal shear strength by Section 4.1.2.4.1.2.3, effects of reinforcement shall be neglected.

4.1.3.4.1 Special Provisions for Shear Wall Specimens with AAC Laid in other than Running Bond: The provisions of Section 4.1.2.4.1.2 shall apply.

4.1.3.5 Flexural Cracking: The flexural cracking strength shall be computed in accordance with Section 4.1.2.6.5.

4.2 Installation:

AAC products are installed in accordance with applicable codes, approved construction drawings, and/or AAC specifications, by installers approved by the manufacturers. A copy of this report, construction drawings, and/or specifications shall be available at the jobsite at all times during installation. The installation instructions within this report shall govern if there are any conflicts with the manufacturer's published installation instructions.

4.3 AAC Masonry Units:

With the exception of the first course, which is laid with Type M, $^1/_4$- to 1-inch-thick (6 to 25 mm) mortar complying with ASTM C 270, AAC wall construction units, manufactured in accordance with ASTM C 1386, are laid with AAC thin-bed or thick-bed mortars described in Section 3.2.3 in accordance with this report, with or without vertical joint mortaring. Where mortar is omitted at vertical joints, the structural drawings shall specifically describe omission based on strength reduction due to unmortared vertical joints as noted in Section 4.1.1.4. The AAC masonry walls are built in running bond, i.e., the vertical joints are staggered a minimum of one quarter the length of the units, but not less than 4 inches (102 mm). The mortar is applied to clean masonry unit surface, using a $^3/_{16}$-inch-by-$^3/_{16}$-inch (4.8 mm by 4.8 mm) V-notched trowel. Unless measures are taken to heat the blocks, the minimum ambient temperature during installation shall be 40˚F (4˚C). When reinforcing is required, cells are to be prepared by the AAC manufacturer or trained installers, and grouted with grout complying with Section 4.1.1.8.1.2. Figure 1 shows reinforcement placement. AAC pier and column construction units are cut to shapes and sizes using hand or electric saws. Ordinary wood working tools can also be used; however, special tools are also available to help maintain accurate lines and levels.

4.4 Special Inspection:

Special inspection of concrete, structural masonry and fastener placement shall conform to Section 1701 of the UBC and Section 1704 of the IBC. The special inspector's duties include verifying panel, masonry unit and mortar identification; panel and unit placement; reinforcement placement for field reinforcement; grout preparation and placement; mortar preparation; and application.

5.0 CONDITIONS OF USE

The Autoclaved Aerated Concrete (AAC) Block Masonry Units described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 The AAC masonry structures are designed using the strength design procedures outlined in this report, and the design shall be in accordance with the applicable code.

5.2 Plans, specifications, engineering calculations and other construction documents specifying the use of autoclaved aerated concrete masonry units shall be submitted to the building official for approval. The calculations and documents shall be prepared by a registered design professional when required by the statues of the jurisdiction where the project is to be constructed.

5.3 Inspection and installation of the AAC masonry units shall comply with the requirements set forth in the applicable code for structural masonry.

5.4 Special inspection shall be provided and shall comply with Section 4.4 of this report.

5.5 The masonry units and mortars are manufactured by the additional listee in this report under a quality control program with inspections by Underwriters Laboratories (AA-668).

6.0 EVIDENCE SUBMITTED

6.1 Descriptive information and installation instructions.

6.2 Data in accordance with the ICC-ES Acceptance Criteria for Concrete Floor, Roof and Wall Systems and Concrete Masonry Wall Systems (AC15), dated June 2003.

6.3. Data in accordance with the ICC-ES Acceptance Criteria for Seismic Design Factors and Coefficients for Seismic-force-resisting Systems of Autoclaved Aerated Concrete (AAC) (AC215), dated October 2003.

6.4 A quality control manual.

6.5 Thermal property data.

7.0 IDENTIFICATION

All AAC product labels include the evaluation report number (ESR-1371), the name of the inspection agency (Underwriters Laboratories), and the following information for field identification.

1. AAC masonry units: All packages of AAC wall masonry units recognized in this report carry the manufacturer's name and/or trademark, a code that indicates the production plant and production date, the product type and the strength class and density in accordance with Table 1 of ASTM C 1386.
2. AAC mortar: The packages of AAC mortar carry the manufacturer's name, mixing instructions, application instructions, and shelf life.

TABLE 1—THERMAL PROPERTIES OF AAC PER INCH OF THICKNESS

CLASS	DRY DENSITY, lb/ft^3 (kg/m^3)	THERMAL CONDUCTIVITY, *K* ($Btu{\cdot}in/hr{\cdot}ft^2{\cdot}F$)	THERMAL RESISTANCE, *R* ($hr{\cdot}ft^2{\cdot}F/Btu$), PER INCH THICKNESS
AAC2	25 (400) ± 1.6	0.80	1.25
AAC3	31 (500) ± 1.6	0.97	1.03
AAC4	37 (600) ± 1.6	0.97	1.03
AAC6	44 (700) ± 1.6	1.25	0.8

For **SI:** $\frac{1\text{Btu}}{\text{hr}\cdot\text{ft}^2\cdot\text{F}} = 1.73\ \frac{M}{m \cdot k}$, $\frac{1\text{hr}\cdot\text{ft}^2\cdot\text{F}}{\text{Btu}} = \frac{0.1761\text{K}\cdot\text{m}^2}{\text{W}}$

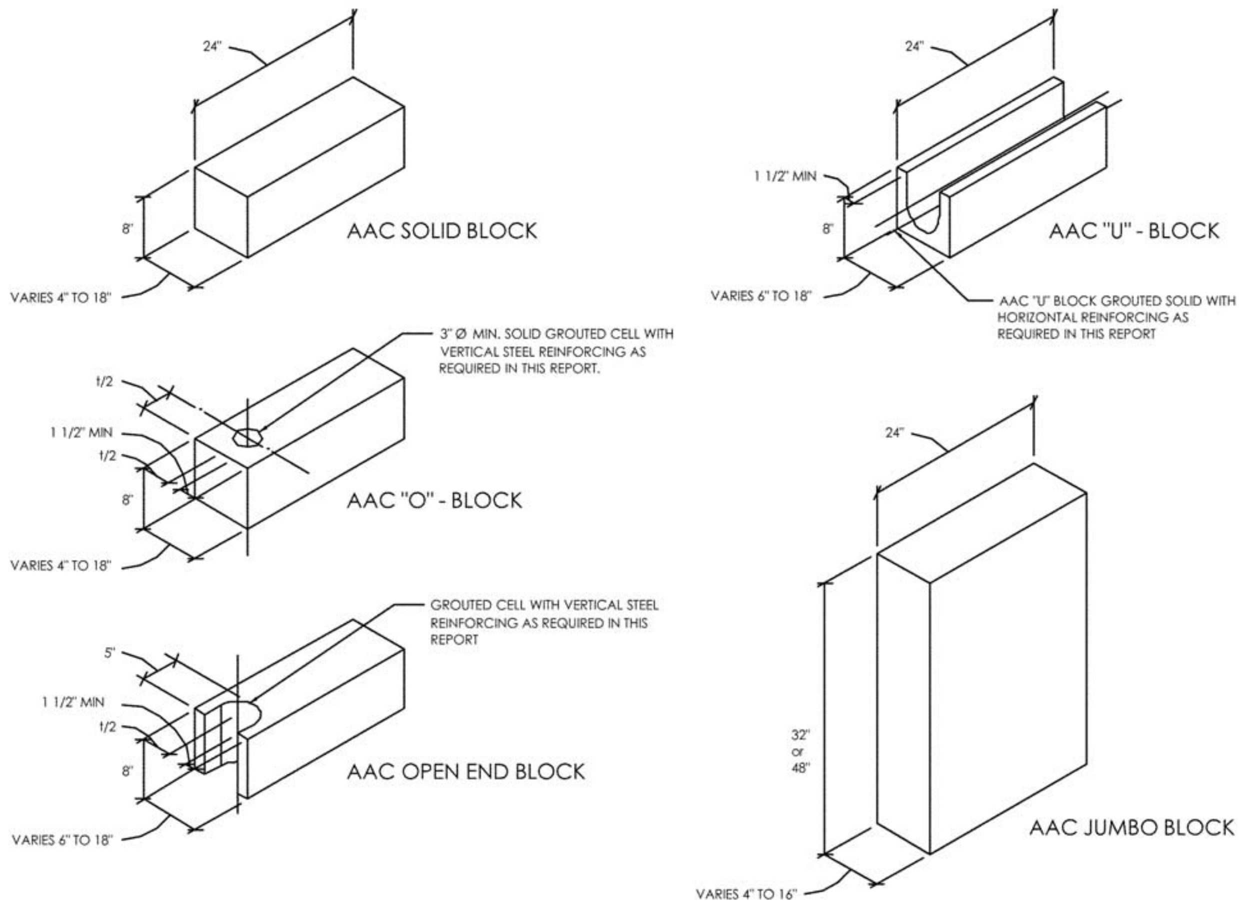

FIGURE 1

TABLE 2—SEISMIC DESIGN COEFFICIENTS AND FACTORS[1]

CODE	BEARING WALL SYSTEM	*R*		Ω_o		C_d	
		$M_u/V_u d_v > 1.0$	$M_u/V_u d_v \le 1.0$	$M_u/V_u d_v > 1.0$	$M_u/V_u d_v \le 1.0$	$M_u/V_u d_v > 1.0$	$M_u/V_u d_v \le 1.0$
IBC	ORMSW	3	$2^1/_2$	1.5	$2^1/_2$	3	$1^3/_4$
	DPMSW	3	2	1.5	$2^1/_2$	3	$1^3/_4$
	OPMSW	3	$1^1/_2$	1.5	$2^1/_2$	3	$1^3/_4$
UBC	ORMSW	5.5	4.5	2.8	2.8	—	—
	DPMSW	5.5	NP	2.8	NP	—	—
	OPMSW	5.5	NP	2.8	NP	—	—

[1]ORMSW = Ordinary reinforced masonry shear wall.
DPMSW = Detailed plain masonry shear wall.
OPMSW = Ordinary plain masonry shear wall.
NP = Not permitted.