

ACI 526R-19

Guide for Design and Construction with Autoclaved Aerated Concrete Panels

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Guide for Design and Construction with Autoclaved Aerated Concrete Panels

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Guide for Design and Construction with Autoclaved Aerated Concrete Panels

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This guide is intended for use by architects, engineers, contractors, building officials, and manufacturers. Its purpose is to present, in a single source, information that can help those individuals design, specify, and construct with factory-reinforced panels of autoclaved aerated concrete (AAC). In this guide, introductory information on AAC is first presented, followed by a description of its manufacture, guidance on structural design using reinforced panels, and guidance on construction with such panels. The body of this guide ends with an extensive background chapter on the material characteristics of AAC and the structural behavior and design of AAC elements.

Keywords: autoclaved aerated concrete; design; sustainability; shear wall.

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CHAPTER 1—INTRODUCTION

1.1—Introduction

Autoclaved aerated concrete (AAC) is produced as masonry type units where it can be reinforced in the field similar to conventional concrete masonry or as factory-produced panels with reinforcement installed in the factory. Reinforcement for factory-installed panels can be supplemented with reinforcing steel as the panels are assembled in the field. Because design and construction provisions already exist for AAC masonry made from masonry-type units without factory-installed reinforcement, this guide touches only briefly on AAC masonry. Provisions for the design and construction of AAC masonry can be found in **TMS 402/602**. This guide addresses design, specification, and construction needs for factory-reinforced panels, for which comparable design and construction provisions do not yet exist.

This guide is also intended to be used as a starting point for the development of mandatory-language design provisions, under the mandate of ACI Committee 318 or other



Fig. 1.3—Macroscopic cellular (void) structure of AAC.

committee so designated by ACI. To facilitate that process, the design provisions proposed in this guide, though written in nonmandatory language as required by ACI, are arranged to follow the format of **ACI 318**.

1.2—Historical background of AAC

AAC was first produced commercially in Sweden in 1923. Since that time, its production and use have spread to more than 40 countries in North America, Central and South America, Europe, the Middle East, Asia, and Australia. This wide experience has produced many case studies of use in different climates and under different building codes.

In the United States, modern uses of AAC began in 1990, for residential and commercial projects in the southeastern states. United States production of plain and reinforced AAC started in 1995 in the Southeast and has since spread to other parts of the country.

1.3—Scope and objectives of this guide

AAC, a form of cellular concrete, is a low-density cementitious product of calcium silicate hydrates in which the low density is obtained by the formation of macroscopic air bubbles, mainly by chemical reactions within the mass during the liquid or plastic phase. The air bubbles are uniformly distributed and are retained in the matrix on setting, hardening, and subsequent curing with high-pressure steam in an autoclave, to produce a homogeneous structure of macroscopic voids, or cells (Fig. 1.3). AAC is broken down into classes defined by the unit weight and compressive strength. Material specifications for this product are prescribed in **ASTM C1693**.

This guide is limited to AAC with a density of 50 lb/ft³ (800 kg/m³) or less.

The specific objectives of this guide are to:

- a) Review the basic characteristics of AAC
- b) Provide a brief history of structural applications of AAC
- c) Review the fabrication of AAC panels
- d) Recommend structural design procedures for factory-reinforced AAC panels
- e) Recommend construction details for use with factory-reinforced AAC panels

The structural design procedures and construction details recommended herein are intended to result in AAC panels with reliable structural capacity, durability, appearance, and overall serviceability.

This guide is limited to AAC with a density of 50 lb/ft³ (800 kg/m³) or less. It is written for structural designers and addresses design using factory-reinforced AAC panels. Design of AAC masonry is also addressed in ACI 530/530.1.

The specific objectives of this guide are to:

- Review the basic characteristics of AAC
- Provide a brief history of structural applications of AAC
- Review the fabrication of AAC panels
- Recommend structural design procedures for factory-reinforced AAC panels
- Recommend construction details for use with factory-reinforced AAC panels

The structural design procedures and construction details recommended herein are intended to result in AAC panels with reliable structural capacity, durability, appearance, and overall serviceability.

Chapter 2 presents standard notation and definitions. Material and thermal characteristics and an introduction to manufacturing are presented in **Chapter 3**. A general design overview is presented in **Chapter 4**. In **Chapter 5**, handling and erection of panels are addressed. **Chapter 6** presents the detailed technical justification for the proposed design provisions. References are presented in **Chapter 7**. **Appendix A** provides design examples. In **Appendix B**, typical design details are used to introduce the reader to specific configurations of the AAC structural elements whose design and construction are addressed by this document.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A	= area of wall, in. ² (mm ²)
A_s	= effective cross-sectional area of horizontal reinforcement in AAC panel, in. ² (mm ²)
d	= distance from centroid of tension steel to fiber at maximum compressive strain (taken as $0.8\ell_w$ for a shear wall), in. (mm)
d_{cross}	= diameter of cross wire in reinforced AAC panel, in. (mm)
d_{long}	= diameter of longitudinal wire in reinforced AAC panel, in. (mm)
E_{AAC}	= modulus of elasticity of AAC, psi (MPa)
f_{AAC}	= tested compressive strength of AAC, psi (MPa)
f'_{AAC}	= specified compressive strength of AAC, psi (MPa)
f_{bond}	= tensile bond strength of AAC, psi (kPa)
f_c	= tested compressive strength of concrete, psi (MPa)
f'_g	= specified compressive strength of grout, psi (MPa)
f_r	= modulus of rupture of concrete, psi (kPa)
f_{rAAC}	= modulus of rupture of AAC, psi (kPa)
f_s	= stress developed in horizontal reinforcement of reinforced AAC panel, psi (MPa)
f_t	= splitting tensile strength of concrete, psi (MPa)

f_{tAAC}	= splitting tensile strength of AAC, psi (MPa)
f_y	= specified yield strength of steel reinforcement, psi (MPa)
h_{crack}	= height of flexural crack in a shear wall at flexure-shear cracking, in. (mm)
h_{wall}	= height of shear wall, in. (mm)
L	= length of a beam or floor slab including the support width, in. (mm)
ℓ_{cross}	= length of transverse steel in AAC panel, in. (mm)
ℓ_n	= clear span of a beam or floor slab, in. (mm)
ℓ_w	= plan length of shear wall, in. (mm)
M	= moment at the base of shear wall, kip-in. (kN-m)
n_{cross}	= number of cross wires
n_{long}	= number of longitudinal wires
P	= axial force acting on wall, kip (kN)
R-value	= thermal resistance of a wall system, h-ft ² °F/Btu (m ² K/W)
S	= section modulus, in. ³ (mm ³)
S_{cross}	= center-to-center spacing of cross wires (transverse steel) in reinforced AAC panel, in. (mm)
s_h	= spacing of longitudinal wires in reinforced AAC panel, in. (mm)
t	= specified thickness of shear wall, in. (mm)
U-value	= thermal conductivity of a wall system, Btu/h-ft ² °F (W/m ² K)
V	= shear at the base of shear wall, kip (kN)
V_{AAC}	= shear strength provided by autoclaved aerated concrete, kip (kN)
V_c	= shear strength provided by concrete, kip (kN)
V_{cr}	= base shear at flexural cracking capacity, kip (kN)
V_{ds}	= strength of an AAC shear wall as governed by crushing of diagonal strut, kip (kN)
V_n	= nominal shear strength of a reinforced concrete section, kip (kN)
V_s	= shear strength provided by the shear reinforcement, kip (kN)
V_{ss}	= sliding shear capacity of AAC shear wall, kip (kN)
w	= horizontal projection of the width of the diagonal strut, in. (mm)
ϵ_{cs}	= drying shrinkage of AAC
μ	= coefficient of friction
ρ_{1693}	= density of AAC as defined by ASTM C1693

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource. Definitions provided herein complement that source.

autoclaved aerated concrete—cementitious product based on calcium silicate hydrates in which low density is attained by the inclusion of an agent resulting in macroscopic voids and is subjected to high-pressure steam curing.

corrosion-inhibiting coating—acrylic synthetic resin material manufactured specifically for coating smooth wire reinforcement embedded in factory-produced AAC panels; embedded wire cages are generally coated by dipping in the liquid coating after the cages are welded.

thin-bed mortar—mortar for use in construction of AAC unit masonry whose joints should not be less than 1/16 in. (1.5 mm).

CHAPTER 3—TYPICAL MATERIAL AND THERMAL CHARACTERISTICS AND MANUFACTURE OF AAC

3.1—Materials used in manufacturing of AAC

Materials for AAC vary with the method of manufacture and the raw materials available at the location of manufacture and are specified in [ASTM C1693](#) and [C1694](#). They include some or all of the following:

- Fine silica sand ([ASTM C33/C33M](#), [C144](#), or [C332](#))
- Class F fly ash ([ASTM C618](#)) with up to 12 percent loss on ignition (LOI)
- Hydraulic cements ([ASTM C150/C150M](#), [C595/C595M](#), or [C1157/C1157M](#))
- Calcined lime ([ASTM C110](#))
- Gypsum ([ASTM C22/C22M](#))
- Expansive agent, such as finely ground aluminum powder or paste
- Mixing water (clean and free of deleterious substances)
- Reinforcement ([ASTM A82](#)), welded to form cages, with corrosion-inhibiting coating

3.2—Typical mechanical and thermal characteristics of AAC

In Table 3.2, typical mechanical and thermal characteristics of AAC are compared with those of conventional concrete, including conventional concrete made with lightweight aggregates. AAC typically has one-sixth to one-third the density and strength of conventional concrete, making it suitable for cladding and infill panels and for load bearing-wall components of low- to medium-rise structures.

The thermal conductivity of AAC is 13 to 16 times smaller than that of conventional concrete, making it energy-efficient. Its fire resistance is slightly longer than that of conventional concrete of the same thickness, making it useful in applications where fire resistance is important. In addition, the fire resistance of AAC is superior to traditional wood construction.

AAC has excellent acoustical properties. Because of its characteristic high internal porosity, AAC has very high sound absorption. Because of its lower density, AAC is not as resistant to sound transmission as conventional concrete of the same thickness. When typical element thicknesses are used, however, AAC has excellent resistance to sound transmission.

3.3—Thermal considerations

One of AAC's more desirable properties is that it provides excellent thermal characteristics for energy-saving construction through a unique combination of relatively high thermal resistance and thermal mass in a single continuous material layer. This material characteristic also helps to reduce air infiltration and mitigate thermal bridging, which often plagues multi-layer envelope assemblies with comparable properties. AAC is typically produced in the density range

Table 3.2—Typical physical characteristics of AAC

Characteristic	AAC	Conventional concrete
Density,* lb/ft ³ (kg/m ³)	25 to 50 (400 to 800)	90 to 150 (1442 to 2400)
Compressive strength f_c , psi (MPa)	290 to 1100 (2.0 to 7.6)	2500 to 10,000 (17.2 to 69)
Moisture content at after autoclaving	30 percent	—
Moisture content in use	5 to 15 percent	—
Coefficient of thermal expansion, /°F (/°C)	4.5×10^{-6} (8.1×10^{-6})	5×10^{-6} (9×10^{-6})
Coefficient of creep, per psi (per MPa)	5×10^{-7} (0.72×10^{-4})	2.5×10^{-7} (0.36×10^{-4})
Drying shrinkage (ϵ_{cs} by ASTM C1693)	200 $\mu\epsilon$	300 to 600 $\mu\epsilon$
Thermal conductivity, Btu-in./ft ² -h-°F	0.7 to 1.3	10 to 20
Fire resistance, h (by ASTM E119)	≤ 4	≤ 4

*Throughout this guide, density is defined consistently with ASTM C1693 because AAC is defined by ASTM C1693. In ASTM C1693, what is referred to as density is actually a unit weight, with units of lb/ft³ (U.S. customary) and units of kg/m³ (old metric). This definition is not strictly correct, nor is it consistent with ACI policy for SI units. It is maintained here for consistency with ASTM C1693.

of 25 to 50 lb/ft³ (400 to 800 kg/m³) and provides an R-value per inch (millimeter) thickness range of 1.3 to 0.7 h-ft²/Btu-in. (9.0 to 4.9 mK/W), respectively, varying inversely with density ([Kosny et al. 2012](#)). At a typical density of 35 lb/ft³ (560 kg/m³), the R-value may be taken conservatively as 1.1 h-ft²/Btu-in. (7.6 mK/W) ([Behrens and Tanner 2008](#)). R-values per unit thickness quantify the steady-state thermal properties and are the inverse of thermal conductivity k . U-values are the thermal conductivity divided by wall thickness to represent a system's conductivity.

AAC's merits are demonstrated through its dynamic heat transfer performance, within the context of its ability to maintain a constant indoor temperature, despite diurnal temperature swings of the exterior environment. This is accomplished by the high material thermal resistance, which reduces the rate of heat conduction flux Btu/h-ft² (W/m²), as well as the relatively high heat thermal mass, a material property that is derived from the material specific heat, density, and thickness ([Aroni 1990](#)). Increasing any of the three latter properties increases the heat capacitance proportionally. Unlike thermal resistance, increasing heat thermal mass does not directly increase the steady-state material resistance to heat flow. Rather, it acts to dampen or delay the transient conduction through the material by requiring that a larger amount of heat from the exterior (source) is used in heating up the material (envelope) before it can increase the interior (sink) surface temperature. Because the amount of heat transferred through the material is proportional to the difference between the surface temperatures, this effectively results in a reduced amount of heat transfer. The result is dynamic mass benefit, where an AAC block wall was dynamically tested by fluctuating the temperature on one surface while measuring the temperature on the other. The

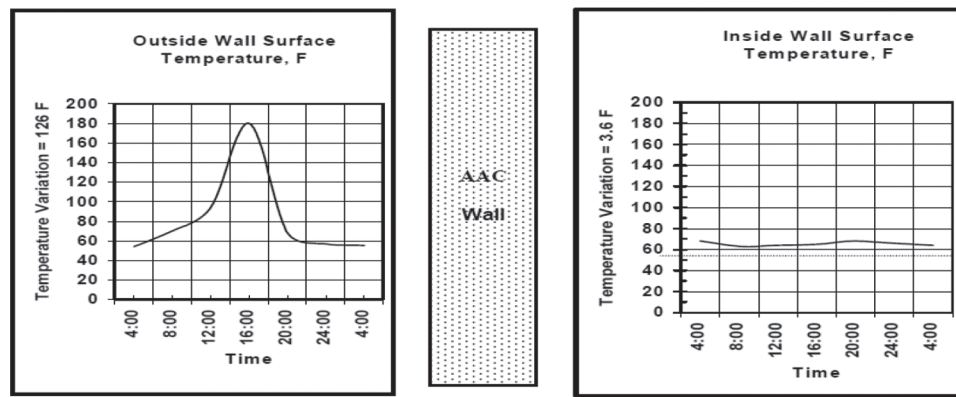


Fig. 3.3—Time-lag thermal mass benefit (Kosny 2000; Kosny et al. 2001). (Note: $1.8^{\circ}\text{F} = 1^{\circ}\text{C}$.)

mass dampening effect on heat transfer is clearly visible by both the reduced amplitude and the hourly delay of the inside versus outside temperature peaks, as illustrated in Fig. 3.3 (Kosny 2000; Kosny et al. 2001).

Although traditional concrete possesses excellent structural qualities, it is a relatively poor insulator. In traditional concrete masonry unit (CMU) construction, some type of supplemental insulation, either through insulated cores or continuous exterior or interior insulation, is required for most climates to provide adequate thermal resistance. The Kosny et al. (2012) study demonstrated that 10 to 12 in. (250 to 300 mm) of AAC block provided roughly equivalent thermal performance to a traditional CMU wall with 2.5 in. (64 mm) of continuous expanded polystyrene insulation board. The same study demonstrated that AAC block significantly outperformed CMU block with insulated cores. This is mostly due to the thermal bridging effect of the CMU block webs, which is not present with solid AAC block.

The dynamic benefit for massive systems (DBMS) value was introduced as a multiplier for the steady-state assembly R-value in a study performed by Oakridge National Laboratory. The DBMS value quantifies and presents an effective R-value for an 8 in. (200 mm) thick AAC wall when whole-building energy analysis considers the dynamic envelope behavior that accounts for AACs heat capacitance versus nonmassive systems such as wood frames (Kosny 2000). DBMS is used to define an effective comparison to the performance of wood-frame systems and is specific to a particular climate. Values ranged between approximately 1.5 in climates with lower diurnal temperature swings (Minneapolis, Miami) to approximately 2.5 in climates with high diurnal temperature swings (Phoenix) where thermal mass benefit is more effectively leveraged. Values are compared to a reference value of 1 for wood-frame walls for all climates. This trend is consistent with the action of thermal mass; the dampening of short-term fluctuations is most effective because the period of the temperature reversal being shorter than the time to reach steady-state allows for an internal reversal of the direction of heat flow prior to conduction reaching its peak. This means that diurnal temperature swings are dampened more effectively, than seasonal temperature fluctuations. The mass benefit is magnified by

the amplitude of the swing. Large daytime-nighttime swings common in desert climates are better suited for thermal mass construction, where short, intense peaks that are dampened by mass would otherwise quickly transmit a large envelope peak through a lightweight construction.

This broader concept of mass benefit has been known and employed since ancient times, as exemplified in the massive adobe structures of the American Southwest and similar construction in desert climates throughout the world. An additional dynamic thermal mass benefit can be found by shifting the peak wall conduction load to a nonpeak hour. If this does not coincide with other cooling loads such as solar gains, lighting, and occupants, the benefits of thermal mass are further magnified. Kosny et al. (2001) performed a similar study for traditional CMU block and demonstrated that the combination of thermal resistance (R-value) and thermal mass intrinsic to AAC is critical. While AAC saw dynamic benefits in all climates as shown by the preceding study, traditional CMU block saw benefit only in Phoenix, which, despite its higher thermal mass, was not as good as AAC. Furthermore, CMU performed worse than wood frame in all other climates, unlike AAC, which outperformed wood frame in all cases.

ASHRAE 90.1 requires that designers demonstrate that an envelope assembly is compliant. The industry-favored approach is a prescriptive compliance path that provides required assembly U-values and insulation R-values based on climate zone that should be met. An overall assembly U-value should be met or continuous insulation should be provided to a certain value, which increases (or decreases in the case of U-value) with climate zone number. Climate Zone 1 is located in the southern most United States and Climate Zone 8 in the extreme northern United States. Buildings are further subdivided by construction type with four categories: mass; metal panel building; steel frame; and wood frame/all others. ASHRAE 90.1 provides a benefit for mass construction by increasing assembly U-values and decreasing insulation requirements for mass construction, which allows for less thermal resistance than nonmassive construction. This mass benefit is most pronounced in cooling dominated southern climate zones such as Zones 1 and 2, where the ratio of mass to non-mass assembly U-value

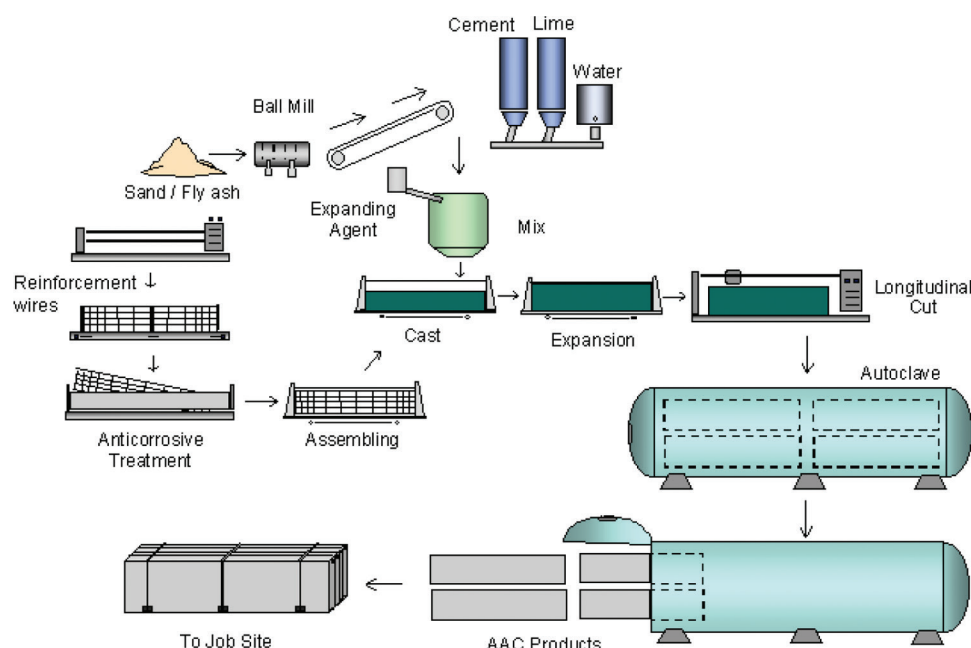


Fig. 3.4—Steps in the manufacture of AAC.

is approximately 6.5:1. As climate zone increases, this ratio decreases until it reaches approximately 1:1 in Climate Zone 7, corresponding to no thermal mass benefit. ASHRAE 90.1 defines a mass wall as a wall with a heat capacity exceeding one of the following two criteria:

- a) 7 Btu/ft²°F (45.2 kJ/m²K)
- b) 5 Btu/ft²°F (32.2 kJ/m²K), provided that the wall has a material unit weight not greater than 120 lb/ft³ (1922 kg/m³)

AAC is not specifically listed by **ASHRAE 90.1**, which includes U-values and heat capacity values for lightweight and normalweight concretes. An area of current research includes both experimental and numerical simulation approaches to provide the necessary performance data to demonstrate AAC code compliance. The goal is to eventually have standardized performance data and parameters incorporated into future codes.

Experimental data have shown AAC with densities ranging from 32 to 35 lb/ft³ (510 to 560 kg/m³) is near the lower limit of this mass wall definition at a thickness of approximately 8 in. (200 mm) and greater. Compliance could be estimated using an assembly U-factor of 0.10 (R-10) that would allow AAC to meet the mass-wall thermal criteria for Climate Zones 4 and below (southern United States), but not for 5 and above (Kosny et al. 2012; Ropelewski and Neufeld 1999). Below this thickness, typical AAC might not provide the required heat capacity or U-factor required for prescriptive compliance. Lighter-density AAC also may fall below the required heat capacitance threshold and would, thus, be categorized as other, subjecting it to the more stringent U-factor requirements of traditionally framed wall systems. This would result in a thicker wall assembly if a prescriptive design path were still desired. The other prescriptive path would be to provide the code-required value of continuous insulation, which would unnecessarily penalize AAC because these values are based on the lesser thermal resistance of normalweight concrete.

Another option for designers of AAC structures would be to perform a whole-building simulation against baseline in accordance with ASHRAE 90.1 using energy modeling software. This is a current option for designers of AAC structures to prove energy code compliance. Energy modeling simulation may also be used to assess energy performance for green building programs such as LEED as well as energy cost benefit and life-cycle cost analysis to provide energy data for design decision making and design optimization of AAC structures.

3.4—Manufacture of AAC

Overall steps in the manufacture of AAC are shown in Fig. 3.4. Because the same basic AAC material can be used for unreinforced AAC units (masonry-type units) as well as reinforced panels, information on both types of units is presented in this chapter. The masonry-type units are addressed only briefly for the sake of completeness. Design and construction provisions for AAC masonry units are not addressed by this document. They are addressed in **ACI 530/530.1**.

3.4.1 Preparation, batching, and mixing of raw materials—Sand is ground to the required fineness in a ball mill, if necessary, and is stored along with other raw materials. The raw materials are then batched and delivered to the mixer. Measured amounts of water and expansive agent are added to the mixer, and the cementitious slurry is mixed.

3.4.2 Casting, expansion, and initial hydration—Steel molds are prepared to receive the fresh AAC slurry mixture. If reinforced AAC panels are to be produced, steel reinforcing cages are secured within the molds. After mixing, the slurry is poured into the molds. The expansive agent creates small, finely dispersed voids in the fresh mixture, which increases the volume by approximately 50 percent in the molds within 1 hour.

3.4.3 Cutting—Within a few hours after casting, the initial hydration of cementitious compounds in the AAC gives



Fig. 3.4.3a—Fresh AAC after removal of molds.



Fig. 3.4.4a—Autoclaving AAC.



Fig. 3.4.3b—Cutting AAC into desired shapes.

it sufficient strength to hold its shape and support its own weight. The material is removed from the molds (Fig. 3.4.3a) and fed into a cutting machine that uses wires to section the blocks and panels into the required sizes and shapes (Fig. 3.4.3b). After cutting, the units remain in their original positions in the larger AAC block.

3.4.4 Autoclaving—After cutting, the aerated concrete product is transported to a large autoclave, where the curing process is completed (Fig. 3.4.4a). Autoclaving is required to achieve the desired structural properties and dimensional stability. The process takes approximately 8 to 12 hours under a pressure of approximately 174 psi (1.20 MPa) and a temperature of approximately 360°F (180°C), depending on the class of material produced. During autoclaving, the wire-



Fig. 3.4.4b—Packaging of finished AAC units.

cut units remain in their original positions in the AAC block. After autoclaving, the individual units are dimensionally stable and are specified to have a drying shrinkage no more than 0.02 percent (**ASTM C1693**). They are then separated for packaging (Fig. 3.4.4b).

3.4.5 Packaging—AAC units are normally placed on pallets for shipping. Unreinforced units are typically shrink-wrapped whereas reinforced elements are typically banded only, using edge guards to minimize localized damage from the banding.

3.4.6 AAC strength classes—AAC is produced in different densities and corresponding compressive strengths, in accordance with **ASTM C1693**. Densities and corresponding strengths are described in terms of strength classes. In each case, the strength class corresponds to the specified compressive strength in psi (MPa) (Table 3.4.6).

Table 3.4.6—Typical material characteristics of AAC in different strength classes*

Strength class	Specified compressive strength, psi (MPa)	Nominal dry bulk density, lb/ft ³ (kg/m ³)	Density limits, lb/ft ³ (kg/m ³)
AAC 2.0	290 (2.0)	25 (400) 31 (500)	22 to 28 (350 to 450) 28 to 34 (450 to 550)
AAC 4.0	580 (4.0)	31 (500) 37 (600) 44 (700) 50 (800)	28 to 34 (450 to 550) 34 to 41 (550 to 650) 41 to 47 (650 to 750) 47 to 53 (750 to 850)
AAC 6.0	870 (6.0)	44 (700) 50 (800)	41 to 47 (650 to 750) 47 to 53 (750 to 850)

*Other strength classes within these ranges and densities may be produced depending on specific design requirements.

Table 3.5.1—Typical dimensions of plain AAC wall units (masonry-type units)

AAC unit type	Width, in. (mm)	Height, in. (mm)	Length, in. (mm)
Standard block	2 to 15 (50 to 375)	8 (200)	24 (610)
Jumbo block	4 to 15 (100 to 375)	16 to 24 (400 to 610)	24 to 40 (610 to 1050)

Table 3.5.2—Typical dimensions of reinforced AAC wall units

Product type	Thickness, in. (mm)	Height or width, in. (mm)	Typical length, ft (m)
Wall panel	2 to 15 (50 to 375)	24 (610)	20 (6)
Floor panel	4 to 15 (100 to 375)	24 (610)	20 (6)
Lintel/beam	4 to 15 (100 to 375)	8 to 24 (200 to 610)	20 (6)

Table 3.6—Dimensional tolerances for reinforced AAC panels (ASTM C1694)

Dimension	Floor or roof panels, in. (mm)	Wall panels, in. (mm)
Length	±0.20 (±5)	±0.20 (±5)
Width	±0.12 (±3)	±0.12 (±3)
Thickness	±0.12 (±3)	±0.12 (±3)
Tongue	±0.12 (±3)	±0.12 (±3)
Groove	±0.12 (±3)	±0.12 (±3)

3.5—Typical dimensions of AAC units

3.5.1 Plain AAC wall units—Typical dimensions for plain AAC wall units (masonry-type units) are shown in Table 3.5.1 and are manufactured based on **ASTM C1691**.

3.5.2 Reinforced AAC panels—Dimensional tolerances, requirements for reinforcement, and other requirements for reinforced AAC panels are specified in **ASTM C1694**. Typical dimensions for reinforced AAC wall units (panels) are shown in Table 3.5.2.

3.6—Dimensional tolerances

In accordance with ASTM C1694, dimensional tolerances for plain AAC wall units are slightly longer for length and the same for thickness and height. All values are summarized in Table 3.6.

3.7—Identification and marking of AAC units

Pallets of unreinforced AAC units should be labeled with strength class, production identification code, and size of units. Reinforced AAC panels should bear product identification information indicating the strength class, production identification code, and a number indicating the specified location in the structure.

CHAPTER 4—STRUCTURAL DESIGN OF REINFORCED AAC PANELS

4.1—Applications of AAC panels

AAC can be used to make unreinforced masonry-type units as well as factory-reinforced floor panels, roof panels, wall panels, lintels, beams, and other special shapes. These elements can be used in a variety of applications, including residential, commercial, and industrial construction. Reinforced wall panels can be used as cladding systems as well as load-bearing and non-load-bearing exterior and interior wall systems. In general, the in-plane shear wall strength is essentially limited to that of plain AAC. As a result, the capacity is considerably smaller than that of conventional concrete, and the end result is that AAC is self-limiting for lateral loads. **ASCE/SEI 7** imposes additional limitations on seismic design categories.

The design recommendations in this chapter are a synthesis of design recommendations from the results of research. In this chapter, the proposed design provisions are briefly introduced.

Loads for structural design of AAC panels should be taken from appropriate standards, such as **ASCE/SEI 7**. Strength reduction factors (ϕ -factors) for AAC panels depend on the actions under consideration. They reflect the statistical variability of the capacity and the accuracy of the capacity-calculation formulas. When failure is governed by yield and fracture of tensile reinforcement, ϕ -factors are identical or lower than those used for reinforced concrete. When failure is governed by crushing or diagonal tension of the AAC itself, ϕ -factors are similar to those used for the design of reinforced concrete. They may even be higher because the factory production of AAC leads to decreased variability in its mechanical characteristics compared with cast-in-place concrete. The design provisions of this chapter are not intended for use with unreinforced masonry-type units because design of those units is covered by **TMS 402/602**.

4.2—Overview of proposed design provisions for reinforced AAC panels

4.2.1 Basic design assumptions—The proposed design provisions for reinforced AAC panels are based on the same principles used for strength design of conventional reinforced concrete elements: strain compatibility between AAC and

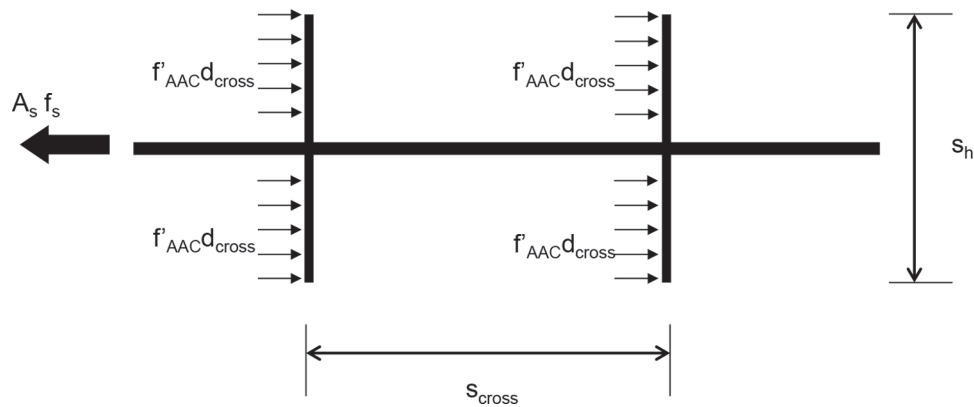


Fig. 4.2.3—Bond mechanism of steel wire cages in AAC (Tanner 2003).

reinforcement (with some modifications as noted), stress-strain behavior of AAC and reinforcement, and equilibrium. The design strength of AAC in compression is based on a specified design compressive strength f'_{AAC} . Compliance with that specified compressive strength is verified by compressive strength testing of AAC cubes using [ASTM C1692](#) when the AAC panels are fabricated. The design strength of AAC in tension is proposed as a function of the specified compressive strength. The design strength of reinforcement in tension is proposed as the specified yield strength.

4.2.2 Combinations of flexure and axial load—AAC panels are designed for combinations of flexure and axial load using principles identical to those for conventional reinforced concrete. Nominal capacity is computed assuming plane sections; strain in tensile reinforcement depends on whether the section is tension- or compression-controlled; the stress in compressive reinforcement is computed based on its strain and its stress-strain behavior; and the distribution of compressive stress in the AAC is approximated by an equivalent rectangular stress block, whose height is $0.85f'_{AAC}$ and whose value of β_1 is 0.67 ([Tanner et al. 2005a,b](#); [Argudo 2003](#)).

The stress in the compressive reinforcement should be neglected in computation of nominal capacity unless top crossties are provided every 6 in. (153 mm). Because reinforced AAC panels usually have equal areas of tensile and compressive reinforcement, flexural capacity is usually tension-controlled. As compression steel is added to a beam, the neutral axis shifts toward the extreme compression fiber. Because the compressive strain in the top fiber is fixed, the strain in the extreme tensile steel increases. By using equal compression and tension steel, the neutral axis shifts to a distance less than two times the clear cover, provided that the compressive stress block accounts for 30 percent of the total compressive force. This results in a strain in the tensile reinforcement of $0.003(d - 2\text{cover}/2\text{cover})$. As long as the ratio of $(d - 2\text{cover}/2\text{cover})$ exceeds $0.005/0.003 = 1.67$, the section will be tension-controlled as defined in [ACI 318](#). For a reinforced panel with depth of 8 in. (200 mm) and cover of 1 in. (25 mm), this ratio is 2.5, which is greater than 1.67. If the panel depth increases, this ratio increases further.

4.2.3 Bond and development of reinforcement—Reinforcement in AAC panels consists of steel wire cages installed when the panels are produced, and deformed reinforcement

installed in 3 to 4 in. (75 to 100 mm) grouted cores as the panels are erected. Bond and development requirements for deformed reinforcement in grout, with sufficient cover, are identical to those used for masonry construction. The limit of bar size to grouted cell size of [ACI 530/530.1](#) ensures that a grouted core will not slip relative to the surrounding AAC ([Tanner et al. 2005a,b](#)).

Bond and development requirements for steel wire cages embedded in AAC are quite different from those for conventional concrete. Because the steel wire is smooth and has a corrosion-resistant coating, the bond strength between the coated wire and the AAC itself is negligible. Bond strength comes from bearing of the cross wires against the AAC. For typical cross wire spacing (s_{cross} in Fig. 4.2.3), local crushing of the AAC under the cross wires can be assumed to redistribute the bearing stresses under the cross wires, leading to a uniform bearing strength of f'_{AAC} under every cross wire. The stresses are initially greater at the weld due to increased stiffness at this location. Weld strength should meet the requirements of [ASTM C1694](#). After local deformations occur at the weld, the more flexible portion of the wires is assumed to resist a uniformly distributed load. This model was confirmed by [Fadaifard \(2008\)](#). Multiplying this stress by the number of cross wires and by the bearing area of each cross wire gives the maximum force that can be developed in the steel wire cage (Fig. 4.2.3). The conservative design assumption is to neglect the contribution of welded wire fabric to the strength of AAC panels subject to reversed cyclic loads such as in applications where the panels are resisting in-plane shear loads in walls subject to wind or seismic loading ([Tanner et al. 2005b](#)). This maximum force in the steel wire cage can limit the flexural capacity of a reinforced AAC panel.

4.2.4 Shear—As with conventional reinforced concrete elements, the shear resistance of AAC panels is computed as the summation of a shear resistance due to the AAC itself (V_{AAC}) and a shear resistance due to reinforcement oriented parallel to the direction of the shear.

Shear resistance of flexural elements is evaluated by modifying ACI 318-14 Eq. (22.5.5.1) using the web-shear cracking approach of ACI 318. The diagonal tension resistance of the AAC is expressed in terms of its specified compressive strength. Principal stresses, including

the effects of axial loads, are equated with this diagonal tension resistance. Solving for VAAC results in the shear resistance of the AAC. A full derivation is included in 6.5.1.

$$\begin{aligned} V_{AAC} &= 0.8\sqrt{f_{AAC}}b_wd \quad (\text{in.-lb}) \\ V_{AAC} &= 0.066\sqrt{f_{AAC}}b_wd \quad (\text{SI}) \end{aligned} \quad (4.2.4a)$$

Shear wall capacity due to the AAC itself (V_{AAC}) is computed using the web-shear approach of ACI 318. The resulting web-shear cracking equation is derived in a similar manner as Eq. (e) of Table 11.5.4.6 of ACI 318-14 was derived. Supporting data is presented in 6.6.1.

$$V_{AAC} = \ell_w t \sqrt{f'_{AAC}} \sqrt{1 + \frac{N_u}{2.4\sqrt{f'_{AAC}}\ell_w t}} \quad (4.2.4b)$$

The shear resistance due to transverse reinforcement embedded in grout is computed based on the cross-sectional area of the transverse reinforcement crossing a hypothetical 45-degree crack in the AAC. As with conventional reinforced concrete, this shear resistance may also be limited by bond and development of the reinforcement.

4.2.5 Bearing—To prevent local crushing of the AAC, nominal stresses in it are limited to $0.65f'_{AAC}$; the leading coefficient is the strength reduction factor for bearing. When AAC floor or roof panels bear on AAC walls, shear failure of the edge of the wall is also possible. This is handled by limiting the shear stress on potential inclined failure surfaces as described in TMS 402/602.

4.2.6 Typical structural details—Typical details for AAC panels are shown in Appendix B. They represent a wide range of typical configurations. Each connection should be designed by qualified structural engineers to consider strength, serviceability, and water penetration. All fasteners should be properly designed and tested as necessary.

CHAPTER 5—HANDLING, ERECTION, AND CONSTRUCTION WITH AAC PANELS

5.1—Handling of AAC panels

AAC panels should be stored on suitably prepared supports. When the panels are stored for an extended period, supports should be placed to minimize any warping due to creep. They should be carefully placed in their final position without being overstressed or damaged. Forces resulting from placing panels should not exceed those resulting from factored loads. Instructions from the manufacturer on how to handle the units should be followed. Special equipment is usually used or recommended by the manufacturer to assist in the transportation and erection of units.

5.2—Erection of AAC wall panels

AAC panels are lifted and placed using specially designed clamps, and are aligned using pry bars with sufficient bearing area to prevent indenting the soft AAC material. Pry bars should be used to slide the panel being placed into the desired final location.

5.2.1 Erection of AAC cladding systems—AAC panels can be used as non-load-bearing cladding systems. This application usually involves horizontally oriented panels attached to steel or reinforced concrete frames. Erection of such panels follows these steps:

- The supporting columns are made plumb and true.
- The bottom panel is set against the supporting columns and over the floor slab or slab-on-ground, on top of a bed joint of conventional masonry mortar. The panel is made true and level.
- Immediately after placing the first panel, the wall anchor plate is fastened to the column (using a dovetail or mechanically locking connector) and the plate is nailed to the AAC panel. The back face of the panel should be flush with the face of the plate anchored to the column.
- Subsequent panels are placed on top of the first one. The top and bottom faces of the horizontal wall panels can have either a tongue-and-groove or a flat profile. Tongue-and-groove joints do not require mortar for structural reasons but may require mortar to prevent leaks. Flat joints are mortared with thin-bed mortar in accordance with ASTM C1660. These standards address minimum dry compressive strength, minimum wet compressive strength, minimum bond strength, minimum open time, and minimum working time. They also require that the mortar provide sufficient bond to the AAC masonry unit so that flexural tensile strength is controlled by the flexural tensile strength of the units rather than by the bond between units and mortar.
- Horizontal and vertical joints are sealed with a flexible sealant resistant to ultraviolet light and ozone exposure. Silicone sealants are most commonly used.

5.2.2 Erection of vertical AAC panels for bearing-wall systems—Vertical AAC panels may also be used as a load-bearing wall system. In such cases, the floor and roof systems are usually designed and detailed as horizontal diaphragms to transfer lateral loads to shear walls. The tops of the panels are connected to the floor or roof diaphragms using a cast-in-place reinforced concrete ring beam. This and many other structural details are addressed in Appendix B.

When vertical wall panels are used in this way, they are set either on flashing, or on a bedding joint of conventional masonry mortar, with or without a waterproofing admixture. Flashing is usually more weather-resistant than mortar for this purpose. Vertical wall panels are usually mortared together with thin-bed mortar.

5.3—Erection of AAC floor and roof panels

AAC floor and roof panels can be erected on concrete, steel, or masonry construction. All bearing surfaces should be level, and minimum required bearing areas (to prevent local crushing) should be maintained.

As in any precast construction system, care should be exercised in the installation of the first panel to ensure correct alignment of the remaining panels. Most floor and roof panels are connected by keyed joints that are reinforced and filled with grout (ASTM C476) to lock the panels together and provide diaphragm action to resist lateral loads.

place reinforced concrete ring beam is normally placed along the perimeter of the diaphragm, completing the system.

5.4—Electrical and plumbing installations in AAC

Electrical and plumbing installations in AAC are placed in routed chases. Care should be taken when laying out chases to maintain the structural integrity of the AAC panels. If reinforcement is cut or AAC is removed, the remaining section should be capable of supporting the design actions, and its calculated deformations and deflections should not exceed permitted values. In vertically spanning AAC elements, horizontal routing should be minimized. In contrast, in horizontally spanning AAC elements, vertical routing should be minimized. When electrical or plumbing installations are large in volume, designated chases are better than routed chases.

5.5—Exterior finishes for AAC

Unprotected exterior AAC deteriorates when exposed to cycles of freezing and thawing while saturated. To prevent such freezing-and-thawing deterioration, and to enhance the aesthetics and abrasion resistance of AAC, exterior finishes should be used. They should be compatible with the underlying AAC in terms of thermal expansion (Table 3.2) and modulus of elasticity, and should be vapor-permeable in accordance with [ASTM C1692](#). Many different types of exterior finishes are available, and the most common are discussed in this guide. Such finishes should be maintained throughout the life of the building.

5.5.1 Polymer-modified stuccos, paints, or finish systems—Polymer-modified stuccos, paints, or finish systems are the most common exterior finishes for AAC. They increase the AAC's water-penetration resistance while allowing the passage of water vapor. Heavy acrylic-based paints containing aggregates are also used to increase abrasion resistance. There is generally no need to level the surface, and horizontal and vertical joints may be chamfered as an architectural feature, or may be filled.

External finishes for AAC walls should be compatible in performance with vapor barriers. In geographic areas where vapor transmission is from outside to inside (typical of the southern United States), moisture in vapor form can condense within the AAC, and should be able to pass in vapor form to the conditioned space. In geographic areas where vapor transmission is from inside to outside (typical of very cold regions), a vapor barrier should be used on the inside face of the AAC wall to control the condensation of moisture inside the AAC.

5.5.2 Masonry veneer—AAC can be used with a wythe of masonry veneer. The veneer is attached to the AAC wall using masonry ties. The space between the AAC and the masonry can be left open (forming a drainage wall), or can be filled with mortar (forming a barrier wall). Drainage walls are generally preferable because they have lower water penetration.

5.5.3 Basement walls—AAC panels are not recommended for use below grade.

5.6—Interior finishes for AAC panels

Interior finishes are used to enhance the aesthetics and durability of AAC. They should be compatible with the underlying AAC in terms of thermal expansion and modulus of elasticity and should be vapor-permeable. Many different types of interior finishes are available, and the most common are discussed herein.

5.6.1 Interior plasters—Interior surfaces of AAC wall panels may have a thin coat of a mineral-based plaster to achieve a smooth, finished surface. Lightweight interior gypsum-based plaster may provide a thicker coating to level and straighten walls and to provide a base for decorative interior paints or wall finishes. Interior plasters have bonding agents to enhance their adhesion and flexibility, and are commonly installed by either spraying or troweling.

5.6.2 Gypsum board—When applied to the interior surface of AAC walls, gypsum board can be attached using metal or pressure-treated wooden furring strips. Moisture-resistant gypsum board can be applied directly to the AAC surface.

5.6.3 High-durability finishes for commercial applications—For commercial applications requiring high durability and low maintenance, acrylic-based coatings are often used. Some contain aggregates to enhance abrasion resistance. High-durability finishes for exterior use should be vapor-permeable and impervious to liquid water.

5.6.4 Ceramic tile—When ceramic wall tile is to be applied over AAC, surface preparation is normally necessary only when the AAC surface requires leveling. In such cases, a portland cement- or gypsum-based parge coat is applied to the AAC surface before setting the ceramic tile. The ceramic tile should then be adhered to the parged wall using either a cement-based thin-set mortar or an organic adhesive. In moist areas, gypsum should not be used. A portland cement-based parge coat should be used for compatibility with the AAC base material and for durability in potentially damp environments, and the ceramic tile should be set with cement-based thin-set mortar only.

CHAPTER 6—TECHNICAL JUSTIFICATION FOR PROPOSED DESIGN PROVISIONS

In this chapter, a synthesis of available data is used to develop a technical justification and design equations for reinforced AAC panels. The provisions are compatible with existing [ACI 318](#) organization, style, and notation, and are consistent with current design provisions for AAC masonry provided in [ACI 530/530.1](#).

Throughout this guide, nominal strengths are proposed as lower fractiles of tested capacities. In ACI design documents, nominal strengths are often based on a lower 5 percent fractile with a 90 percent confidence interval for a normal distribution (for the given sample size, there is a 90 percent probability that only 5 percent of the strengths will lie below the selected nominal strength). This principle is generally followed in this guide.

In a few cases, other fractiles are used. Each is discussed in the following, with appropriate justification.

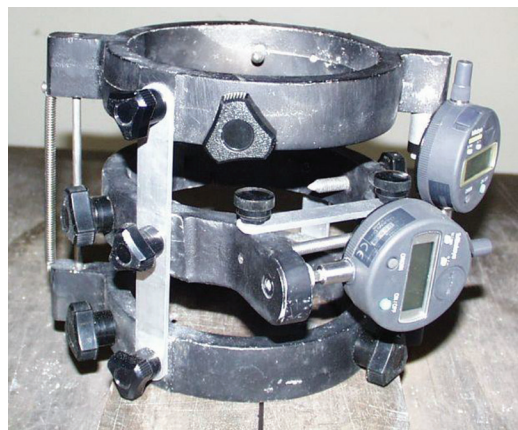


Fig. 6.1.2a—Test setup for measuring compressive stress versus strain (Tanner 2003).

6.1—Key mechanical characteristics of AAC

6.1.1 Compressive strength of AAC—Because AAC was developed in Europe, compressive strength of AAC is generally measured by testing 4 in. (100 mm) cubes. **ASTM C1693** lists the required compressive strength for each class of AAC. Within the United States, compressive strengths are evaluated based on compression cylinders. In general, concrete cube tests have a higher apparent strength than cylinders due to confinement from the loading platens. Compressive strength testing conducted on cubes and cores from the same parent AAC indicated that results of cubes and cylinders provides similar results (Tanner 2003). The cellular structure of AAC reduces confinement at the top and bottom of cubes.

Historically, AAC material properties were determined by using a linear relationship with density; **Argudo (2003)** compiled a suite of material tests and reported on the presence of outliers. This was corroborated by **Chen et al. (2013)**. However, when key mechanical properties such as modulus of elasticity and tensile strengths were plotted using compressive strength as the independent variable, the outliers fell into place. In this document, mechanical properties are plotted as a function of compressive strength, which is consistent with **ACI 318**.

6.1.2 Stress-strain behavior and modulus of elasticity of AAC—Tests have been performed to measure the strains up to failure of 4 in. (100 mm) diameter AAC cores (Tanner 2003; **Cancino 2003**). Each data point represents the average of six tests. Strains were measured with a set of rings attached to the cylinder (Fig. 6.1.2a). Once the specimen begins to lose strength quickly, the reported strains may be smaller than the actual value due to limitations of the scanning rate of the data acquisition system.

The stress-strain curves for each specimen are presented in Fig. 6.1.2b through 6.1.2d. Typical classes of AAC had compressive strengths greater than 0.6 psi (4.1 KPa) reached strains of 0.003—this is the proposed design value. However, some of the lower classes of AAC resulted in lower maximum useful strains, values of 0.0012 and 0.0021 are recommended for Classes 2 and 3 AAC, respectively.

6.1.2.1 Discussion of results for modulus of elasticity of AAC—Modulus of elasticity tests performed on prisms with

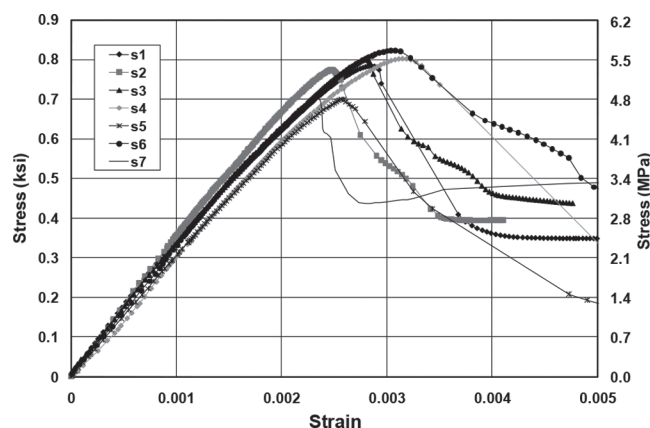


Fig. 6.1.2b—Compressive stress versus strain for CO-S 1 (Tanner 2003).

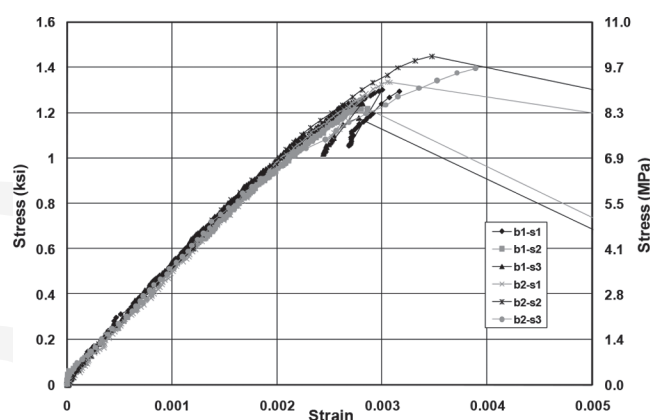


Fig. 6.1.2c—Compressive stress versus strain for HE-S2 (Tanner 2003).

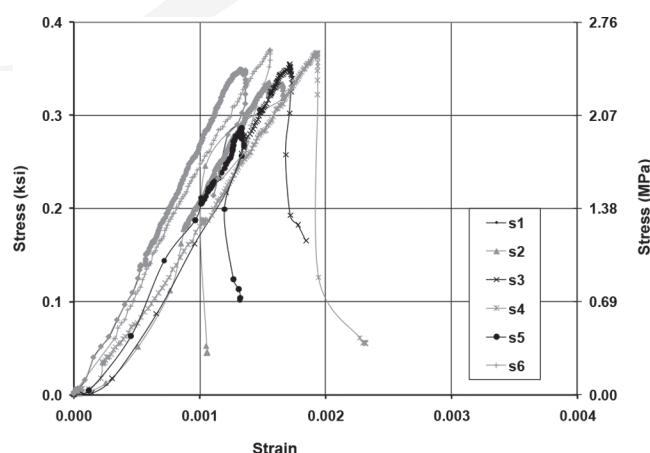


Fig. 6.1.2d—Compressive stress versus strain for BA (Cancino 2003).

strains reported up to a stress of $0.4f_{AAC}$ (**Dembowski 2001; Fouad and Dembowski 2005a; Snow 1999**). Compression tests were performed on four 4 x 8 in. (100 x 200 mm) cores by **Construction Technology Laboratories (1999)** to determine the compressive strength and modulus of elasticity. Results for the expanded set of data are presented in Tables 6.1.2.1a and 6.1.2.1b. Data for loading applied parallel and perpendicular to the direction of ribs (Fig. 6.1.2.1) are similar.

Table 6.1.2.1a—Modulus of elasticity for load perpendicular to the direction of rise (Argudo 2003; Tanner 2003; Dembowski 2001; Snow 1999)

Data Source	Manufacturer	Measured E , ksi (GPa)	COV, %, for E	Measured f_{AAC} , psi (MPa)	COV, %, for f_{AAC}
UAB	Hebel	186 (1.28)	39.5	290 (2.00)	15
	Ytong	310 (2.14)	4.9	600 (4.14)	5.3
	Contec	294 (2.03)	4.9	570 (3.93)	11.5
UT Austin	Contec 1	323 (2.23)	7.9	781 (5.38)	5.6
	Contec 2	424 (2.92)	2.7	1040 (7.17)	4.7
	Babb 1	462 (3.19)	7.1	1140 (7.86)	3.8
	Hebel 2	511 (3.52)	9.3	1330 (9.17)	6.6
	Ytong 2	269 (1.85)	3.4	650 (4.48)	6.8
	Babb 2	271 (1.87)	11.7	496 (3.42)	6
CTL	ACCOA	460 (3.17)	13.6	913 (6.29)	5.6

Table 6.1.2.1b—Modulus of elasticity for load parallel to the direction of rise (Dembowski 2001; Snow 1999)

Manufacturer	E , ksi (GPa)	COV, %	f_{AAC} , psi (MPa)	COV, %
Hebel	139 (0.96)	18	177 (1.22)	17.3
Ytong	278 (1.92)	29	467 (3.22)	13
Contec	271 (1.87)	9	450 (3.10)	12

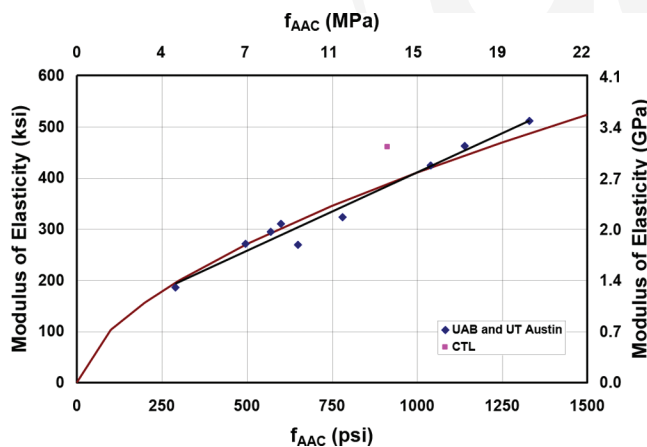


Fig. 6.1.2.1—Modulus of elasticity versus compressive strength for loading parallel to the direction of rise (Argudo 2003; Tanner 2003; Dembowski 2001; Snow 1999).

with a trend of 25 to 50 ksi (172 to 350 MPa) higher for loading perpendicular to the direction of rise (Argudo 2003).

Figure 6.1.2.1 illustrates the modulus of elasticity versus compressive strength. Based on the available data, a reasonable curve fit is a nonlinear equation that passes through the origin. The squared correlation coefficient R^2 is 0.97.

$$\begin{aligned} E_{AAC} &= 6500 f_{AAC}^{0.6} \quad (\text{in.-lb}) \\ E_{AAC} &= 887.8 f_{AAC}^{0.6} \quad (\text{SI}) \end{aligned} \quad (6.1.2.1)$$

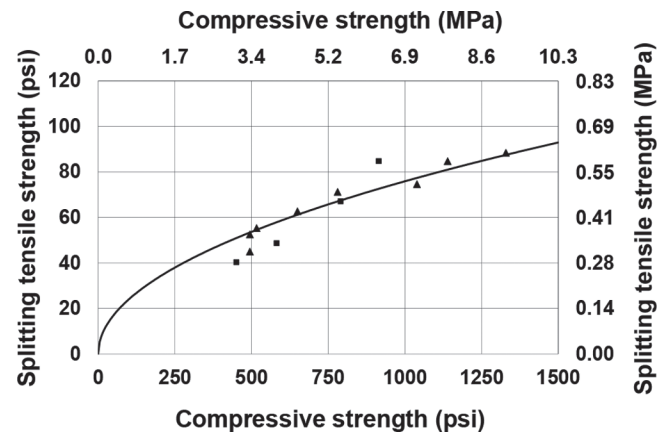


Fig. 6.1.3.1a—Additional test data to evaluate splitting tensile strength (Parker et al. 2007).

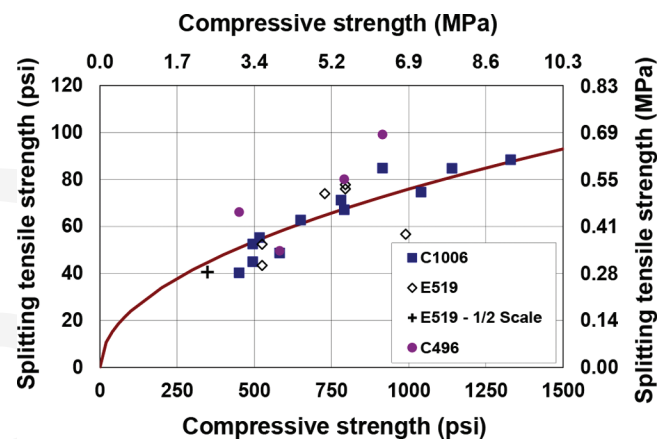


Fig. 6.1.3.1b—Various ASTM tensile strengths versus compressive strength (Tanner et al. 2011; Parker et al. 2007).

where f_{AAC} and E are in psi (MPa).

6.1.3 Tensile strength of AAC—Tensile strength testing of AAC includes both splitting tensile strength and modulus of rupture. Each is discussed in the following sections.

6.1.3.1 Splitting tensile strength of AAC—Data for splitting tensile strengths of material were obtained on AAC blocks according to **ASTM C1006**, using specimens oriented so that splitting tensile stresses acted parallel to the direction of rise (Tanner 2003). Each data point represents the average of four to eight specimens. The direction of rise refers to the direction that the green AAC increases in volume. Parker et al. (2007) performed additional splitting tensile strength and this compilation is presented in Fig. 6.1.3.1a. The resulting Eq. (6.1.3.1) provides good correlation with additional experimental work. Furthermore, the resulting expression is also consistent with **ACI 318** because the tensile strength is expressed as a function of the square root of the specified compressive strength

$$\begin{aligned} f_t &= 2.4 \sqrt{f_{AAC}} \quad (\text{in.-lb}) \\ f_t &= 0.2 \sqrt{f_{AAC}} \quad (\text{SI}) \end{aligned} \quad (6.1.3.1)$$

where f_t and f_{AAC} are in psi (MPa).



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Table 6.1.3.2a—Modulus of rupture: Methods 1 and 2 (Dembowski 2001; Snow 1999)

Material	Tensile stresses parallel to the direction of rise	Tensile stresses perpendicular to the direction of rise	Measured compressive strength f_{AC} , psi (MPa)
	Modulus of rupture f_r , psi (MPa)	Modulus of rupture f_r , psi (MPa)	
HE-1	115 (0.79)	96 (0.66)	280 (1.93)
HE-2	208 (1.43)	167 (1.15)	560 (3.86)
HE-3	331 (2.28)	211 (1.45)	910 (6.27)
YT-1	113 (0.78)	87 (0.60)	330 (2.28)
YT-2	217 (1.50)	177 (1.22)	630 (4.34)
YT-3	115 (0.79)	106 (0.73)	400 (2.76)
CO-1	135 (0.93)	131 (0.90)	320 (2.21)
CO-2	174 (1.20)	159 (1.10)	450 (3.10)
CO-3	225 (1.55)	157 (1.08)	690 (4.76)

An analytical study was conducted by Parker et al (2007) to compare and contrast other methods to evaluate the tensile strength of AAC. Later, the data set was modified to include a wider set of data points. This is presented in Fig. 6.1.3.1b, and Eq. (6.1.3.1) correlates well with a more robust set of data.

6.1.3.2 Relationship between modulus of rupture and splitting tensile strength—A compilation of data on modulus of rupture are reported in Tables 6.1.3.2a and 6.1.3.2b (Fouad and Dembowski 2005a). These tables refer to Method 1 and Method 2. Method 1 is a RILEM method involving midpoint loading with a shear span-to-depth ratio (a/d) of 1.25 (RILEM Technical Committees 78-MCA and 51-ALC 1993). Method 2 is a modified ASTM C78/C78M method with two third-point loads and an a/d of 1.75 (Dembowski 2001; Snow 1999). A relationship between the splitting tensile strength and modulus of rupture was determined by using Eq. (6.1.3.1) and then compared to the modulus of rupture.

Table 6.1.3.2c shows values of modulus of rupture reported by Fouad and Dembowski (2005a) for different classes and corresponding oven-dry densities of AAC. Using Eq. (6.1.3.1), the splitting tensile strength of those specimens was also calculated. The average ratio between the reported modulus of rupture and the calculated splitting tensile strength is 2.26, with a coefficient of variation (COV) of 19.1 percent (Eq. (6.1.3.2a)). For design purposes, a ratio of 2.0 is proposed (Eq. (6.1.3.2b)) (Argudo 2003).

$$f_r = 2.26f_t \quad (6.1.3.2a)$$

$$f_r = 2f_t \quad (6.1.3.2b)$$

6.1.3.3 Modulus of rupture of AAC using combined data—

In this section, results from modulus of rupture tests are compared to results from splitting tensile tests (Fouad and Dembowski 2005a; Argudo 2003). For purposes of comparison, the latter are multiplied by the average value of 2.26, developed as explained previously and used in Eq. (6.1.3.2a).

Table 6.1.3.2b—Modulus of rupture with tensile stresses parallel to the direction of rise with span to depth ratio of Method 2 (Dembowski 2001; Snow 1999; Tanner 2003)

Material	Modulus of rupture f_r , psi (MPa)	Measured compressive strength f_{AC} , psi (MPa)
HE-1	56.0 (0.39)	280 (1.93)
HE-2	86.0 (0.59)	560 (3.86)
HE-3	154.0 (1.06)	910 (6.27)
YT-1	60.0 (0.41)	330 (2.27)
YT-2	106.0 (0.73)	630 (4.34)
YT-3	96.0 (0.66)	400 (2.76)
CO-1	79.0 (0.54)	320 (2.21)
CO-2	91.0 (0.63)	450 (3.10)
CO-3	127.0 (0.88)	690 (4.76)
UT-CO-1	169.5 (1.17)	781 (5.38)
UT-CO-2	177.5 (1.22)	1040 (7.17)
UT-BA-1	201.6 (1.39)	1140 (7.86)
UT-HE-2	210.4 (1.45)	1330 (9.17)
UT-YT-2	149.2 (1.03)	650 (4.48)
UT-YT-1	131.6 (0.91)	517 (3.56)

Table 6.1.3.2c—Ratios between measured modulus of rupture at ASTM C1693 density and splitting tensile strength estimated using oven-dry density for different classes of AAC (Argudo 2003; Tanner 2003)

Oven-dry density, lb/ft ³ (kg/m ³)	Measured f_r , psi (MPa)	f_r/f_t
32.0 (2.0)	123.0 (0.85)	2.04
38.0 (2.4)	202.0 (1.39)	2.75
42.0 (2.6)	162.0 (1.12)	1.97

Modulus of rupture results are presented separately for Methods 1 and 2. Some Method 1 tests were performed with specimens oriented so that flexural tensile stresses acted parallel to the direction of rise and others were performed so that stresses acted perpendicular to the direction of rise. Method 2 tests were performed with specimens oriented so that flexural tensile stresses acted parallel to the direction of rise (Fouad and Dembowski 2005a) (Table 6.1.3.2a and 6.1.3.2b) and are compared in Fig. 6.1.3.3a and 6.1.3.3b to the corresponding calculated modulus of rupture results (factor of 2.26), from splitting tensile tests reported by Tanner (2003). The best single linear regression occurs with results from Method 2. This is reasonable because the relationship of Eq. (6.1.3.2a) was derived based on Method 2 tests, and the flexural tensile stresses act parallel to the direction of rise in both the ASTM C1006 tests and the Method 2 tests.

Regression lines of Fig. 6.1.3.3a through Fig. 6.1.3.3c were calculated for all data available. Those graphs also contain the RILEM relationship between modulus of rupture and compressive strength expressed in Eq. (6.1.3.2).

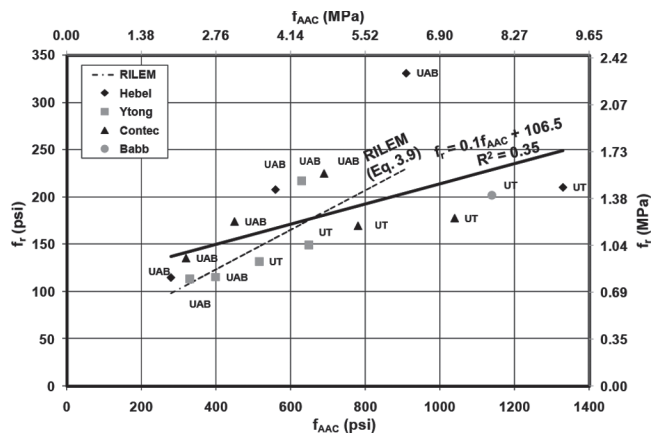


Fig. 6.1.3.3a—Modulus of rupture versus compressive strength for Method 1 (stresses parallel to direction of rise) (Argudo 2003; Dembowski 2001; Snow 1999).

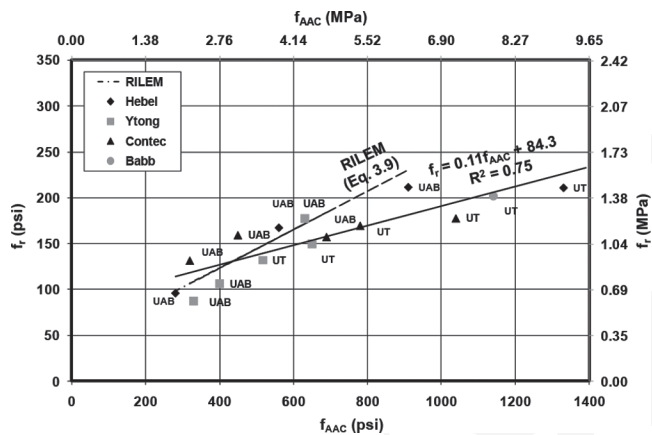


Fig. 6.1.3.3b—Modulus of rupture versus compressive strength for Method 1 (stresses perpendicular to direction of rise) (Argudo 2003; Dembowski 2001; Snow 1999).

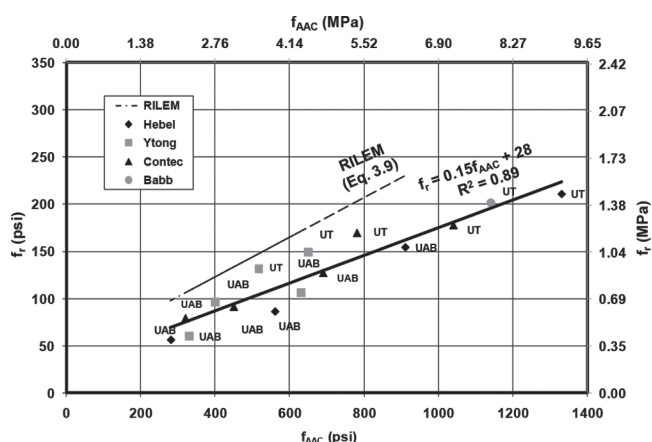


Fig. 6.1.3.3c—Modulus of rupture versus compressive strength for Method 2 (stresses parallel to the direction of rise) (Argudo 2003; Dembowski 2001; Snow 1999).

$$f_r = 0.27 + 0.21 \cdot f_{AAC} \quad (6.1.3.3)$$

where f_{AAC} and f_r are in MPa.

The RILEM Technical Committees 78-MCA and 51-ALC (1993) equation approximates the observed relationship for

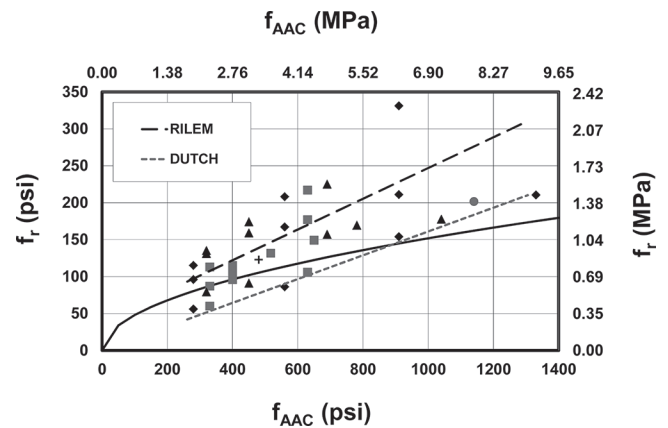


Fig. 6.1.3.4—Relationship between f_r and f_{AAC} —Method 2 (Argudo 2003; Tanner 2003; Dembowski 2001; Snow 1999; Tanner et al. 2011).

Method 1, with stresses parallel to the direction of rise; it is unconservative for Method 1 with stresses perpendicular to the direction of rise, as well as for Method 2. In addition, it does not pass through the origin of the graph. Modulus of rupture data from tests using Method 1 with flexural tensile stresses oriented parallel to the direction of rise are approximately 1.25 times the corresponding results for modulus of rupture tests with flexural tensile stresses oriented perpendicular to the direction of rise. It can be concluded that AAC is stronger in flexural tension when tested so that the flexural stresses are oriented parallel to the direction of rise. Modulus of rupture data from Method 1 are almost twice as high as those obtained using Method 2 when tensile stresses act parallel to the direction of rise. This is attributed to the a/d relationship used in those tests. Specimens tested under Method 1 failed predominantly in flexure whereas those tested under Method 2 failed in a combination of flexure and shear.

6.1.3.4 Relationship between modulus of rupture (f_r) and compressive strength (f_{AAC})—As proposed in Eq. (6.1.3.2b), the modulus of rupture (f_r) can be expressed as twice the splitting tensile strength (f_t). Substituting Eq. (6.1.3.2b) into Eq. (6.1.3.1), Eq. (6.1.3.4a) is obtained, giving the modulus of rupture as a function of the compressive strength (Argudo 2003; Tanner 2003).

$$f_r = 4.8\sqrt{f_{AAC}} \quad (\text{in.-lb}) \quad (6.1.3.4a)$$

$$f_r = 0.4\sqrt{f_{AAC}} \quad (\text{SI})$$

where f_{AAC} and f_r are in psi (MPa).

As discussed in 6.1.3.3, the best correlation between f_r and f_{AAC} occurs for Method 2. Figure 6.1.3.4 shows data for Method 2, along with RILEM relationship (Eq. (6.1.3.3), Eq. (6.1.3.3a)), and the relationship proposed by the Netherlands Normalisatie-Instituut (1987) (Eq. (6.1.3.4b)).

$$f_r = 0.15f_{AAC} \quad (6.1.3.4b)$$

where f_{AAC} and f_r are in psi.

While the RILEM Technical Committees 78-MCA and 51-ALC relationship overestimates f_r for more than half of

Table 6.1.4—Calculated modulus of rupture of AAC shear walls (Tanner et al. 2005a,b)

Specimen	Axial load, kip (kN)	First tested V_{cr} , kip (kN)	Second tested V_{cr} , kip (kN)	First calculated f_{bonds} , psi (MPa)	Second calculated f_{bonds} , psi (MPa)
3	120 (534)	65.9 (293)	78.9 (351)	67.1 (0.47)	92.6 (0.64)
4	120 (534)	90.0 (400)	71.5 (318)	117.9 (0.82)	80.9 (0.56)
5	60 (267)	53.9 (240)	54.5 (243)	74.8 (0.52)	75.9 (0.53)
7	80 (356)	29.4 (131)	30.8 (137)	91.1 (0.63)	98.8 (0.69)
9	60 (267)	12.4 (55)	11.4 (51)	77.5 (0.54)	64.7 (0.45)
11	25 (111)	6.8 (30)	4.9 (22)	269.5 (1.87)	175.7 (1.22)
13	25 (111)	7.0 (31)	5.9 (26)	111.7 (0.78)	88.0 (0.61)
14b	5 (22)	2.9 (13)	2.9 (13)	76.5 (0.53)	76.5 (0.53)
15a	5 (22)	8.5 (38)	10.6 (47)	40.3 (0.28)	55.8 (0.39)
15b	25 (111)	7.8 (35)	8.3 (37)	35.1 (0.24)	38.8 (0.27)
16	25 (111)	13.1 (58)	2.4 (11)	74.2 (0.52)	NA
Assemblage	30 (134)	40.8 (0)	36.5 (162)	62.7 (0.44)	54.6 (0.38)
				Mean	68.3 (0.471)
				COV, %	33

the data points, the Netherlands Normalisatie-Instituut (1987) relationship underestimates it for the entire range of values of f_{AAC} . Equation (6.1.3.4a) provides the best estimates for low- and medium-strength materials and becomes conservative for high-strength materials.

6.1.4 Tensile bond between AAC and bedding mortar—Flexural cracking was observed in 11 shear-wall specimens and one full-scale assemblage specimen (Tanner et al. 2005a,b; Varela et al. 2006). Flexural cracking is governed by the modulus of rupture of the AAC, or by the tensile bond strength across a leveling bed joint if such a joint is present in the element under consideration. In all cases, flexural cracks formed between the AAC and the masonry leveling bed, indicating tensile bond failure between the two materials. Table 6.1.4 compiles relevant test data and presented the flexural bond strength.

The mean modulus of rupture is 68 psi (0.47 MPa), and the corresponding 20 percent lower fractile is 49.4 psi (0.341 MPa). The larger lower fractile is justified because flexural cracking at the base of a shear wall is expected for moderate lateral loads. On this basis, the proposed design value for modulus of rupture should not exceed 50 psi (0.345 MPa) if a leveling bed joint is present in the AAC element

This section presents a combined summary of data from Fouad and Dembowski (2005a) and Tanner et al. (2005b). Fouad and Dembowski (2005a) report tensile bond strengths of 13 and 15 psi (90 and 103 kPa) for mortar meeting ASTM C270 Types M and S, respectively. The bond strengths were improved to 43 and 47 psi (296 and 324 kPa) by adding thin-bed mortar to the ASTM C270 mortar in a 1:1 proportion. Those strengths are lower than the average reported in Tanner (2003) (68.3 psi [471 kPa]). In the shear-wall specimens at UT Austin, each panel was cleaned and prewetted prior to placing the panel on the leveling mortar. No preparatory measures were reported by Dembowski (2001). The difference in bond strength implies that tensile bond strength

Table 6.1.5—Modulus of rupture of AAC specimens with thin-bed mortar joints with tensile bond strengths parallel to the direction of rise: Method 2 (Argudo 2003; Dembowski 2001; Snow 1999)

Material	Measured density ρ_{1693} , lb/ft ³ (kg/m ³)	Modulus of rupture f_r , psi (MPa)
HE-1	26 (412)	60 (0.41)
YT-1	29 (464)	71 (0.49)
YT-2	40 (635)	85 (0.59)
YT-3	44 (706)	39 (0.27)
CO-1	31 (502)	80 (0.55)
CO-2	35 (557)	87 (0.60)
CO-3	39 (629)	101 (0.70)

is sensitive to construction practices. If a higher value of bond strength is to be used, the construction practices should be carefully defined (Dembowski 2001; Argudo 2003).

6.1.5 Tensile bond between AAC and thin-bed mortar—Tests to evaluate modulus of rupture in AAC specimens with thin-bed mortar joints were completed based on Method 2 described in 6.1.3.2 (Dembowski 2001; Snow 1999). Data are presented in Table 6.1.5 and plotted in Fig. 6.1.5 where the shear strength tests on solid specimens and with thin-bed mortar joints are referred as AAC and JOINT.

Modulus of rupture tests from specimens with thin mortar joints showed tensile bond failures of the AAC material for low-strength material only. As the strength of the AAC increased, failure was governed by the tensile bond strength of the thin-bed mortar. If the failure is governed by the material, the slope would be the same as the modulus of rupture. Figure 6.1.5 shows, however, that as the density increases, the failure is governed by the thin-bed mortar joint. The limiting tensile bond strength is 80 psi (552 kPa) for materials with f_{AAC} between 450 and 700 psi (3.10 and 4.83 MPa).

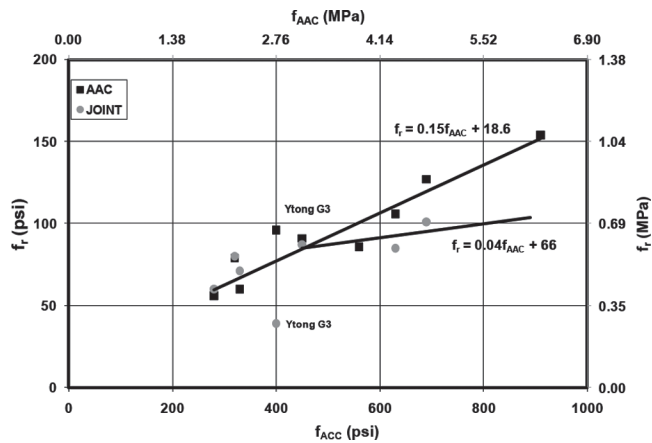


Fig. 6.1.5—Tensile bond strength versus compressive strength (Argudo 2003; Dembowski 2001; Snow 1999).

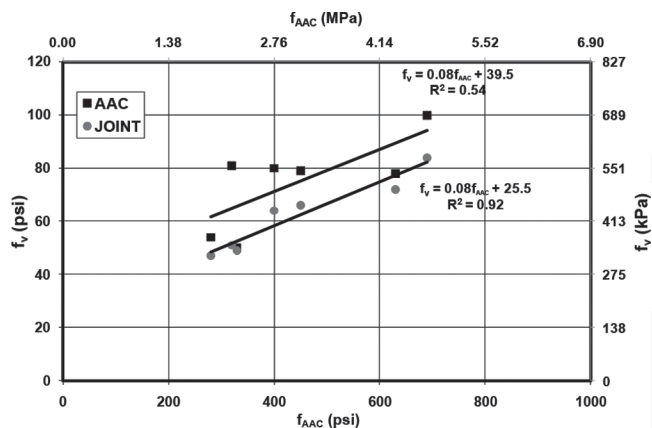


Fig. 6.1.6a—Shear strength of AAC units and thin-bed mortar versus compressive strength (Argudo 2003; Dembowski 2001; Snow 1999).

For materials with f_{AAC} less than 450 psi (3.10 MPa), the AAC unit failure governs, regardless of the type of mortar used and the extent of polymer modification by the addition of thin-bed mortar.

Flexure-dominated shear wall specimens exhibited flexural shear cracking that initiated in the bed joints. In each test, the flexural portion of flexure-shear cracks formed at the horizontal joints between modular blocks, rather than in the AAC material (Tanner 2003).

The requirement that the tensile bond behavior of AAC be governed by the material itself, rather than the mortar, is crucial to the development of design equations, testing requirements, and maximum limitations on the spacing of prescriptive reinforcement. It is essential to verify that with currently specified mortars, tensile bond strength is governed by the AAC material itself.

6.1.6 Shear bond between AAC and thin-bed mortar—Direct shear tests were conducted to evaluate the shear strength in AAC units with and without thin-bed mortar joints. Three AAC units were joined by mortar and tested in flexure using a modified **ASTM C78/C78M** method. Two-point loads were applied near supports to reduce the moment

Table 6.1.6a—Shear tests on AAC with and without thin-bed mortar (Dembowski 2001; Snow 1999)

Manufacturer's specifications	Specimens without thin-bed mortar joints	Specimens with thin-bed mortar joints
Material	Shear strength f_v , psi (MPa)	Shear strength f_v , psi (MPa)
Hebel-HG1	54 (0.37)	47 (0.32)
Ytong-YG1	50 (0.34)	49 (0.34)
Ytong-YG2	78 (0.54)	72 (0.50)
Ytong-YG3	80 (0.55)	64 (0.44)
Contec-CG1	81 (0.56)	51 (0.35)
Contec-CG2	79 (0.54)	66 (0.46)
Contec-CG3	100 (0.69)	84 (0.58)

and maximize the shear in the specimen (Dembowski 2001; Snow 1999). The a/d was 0.13.

Data obtained for AAC specimens tested in shear with and without thin-bed mortar joints are compared in terms of shear strength f_v versus compressive strength f_{AAC} , as shown in Fig. 6.1.6a, where the shear strength tests on specimens without and with joints are referred as AAC and JOINT, respectively.

Data for the shear strength in AAC units and specimens with thin-bed mortar joints are presented in Table 6.1.6a. Comparison of results between AAC specimens with and without thin-bed mortar leads to the following conclusions:

- The shear bond strength in the unit-mortar joint can become the controlling mechanism of failure for all the AAC materials tested regardless of their compressive strength or density.

- The shear strengths of AAC specimens with and without thin-bed mortar joints increase as the compressive strength and density of the material increase. Therefore, in the low- to medium-strength AAC, joint shear failures are related to the strength of the AAC material itself, provided that thin-bed mortar bond strength exceeds the shear strength of AAC.

To determine the shear capacity between thin-bed mortar and AAC, Tanner (2003) and Varela (2003) conducted 11 direct shear tests on AAC specimens with thin-bed mortar joints. Their direct shear specimen is shown in Fig. 6.1.6b and 6.1.6c. The specimen height was 33 in. (840 mm) and the joint length was 15 in. (380 mm). Two batches of AAC were used in the construction of the direct-shear specimens with thin-bed mortar only. The notation for each specimen is DS-TB-1 (DS = direct-shear specimen; TB = thin-bed mortar; 1 = Specimen 1).

The peak load and maximum stress for each specimen are presented in Table 6.1.6b. The average shear stress for each joint is the total load divided by the area of the joint. The shear strength is based on the maximum load resisted by each specimen. The mean shear strength is 72 psi (0.5 MPa) with a COV of 40 percent.

Due to the large variation in the test results for the direct-shear strength, it was suspected that the thin-bed mortar may be sensitive to the mixing process. Additional specimens were tested with two batches of thin-bed mortar mixed by

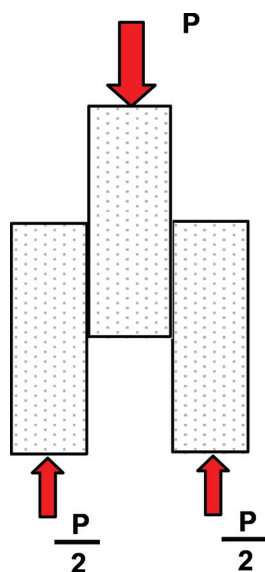


Fig. 6.1.6b—Elevation of direct-shear specimens (thin-bed mortar only).

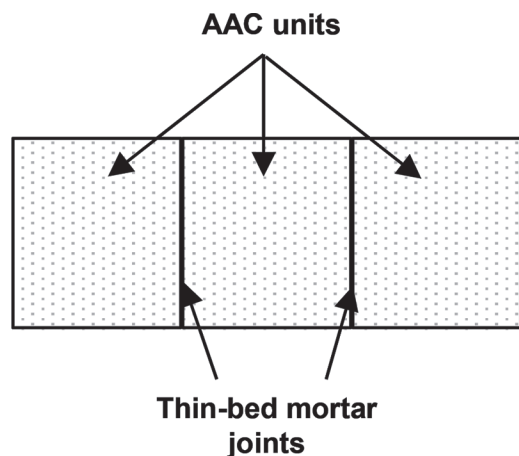


Fig. 6.1.6c—Plan view of direct-shear specimens (thin-bed mortar only).

different individuals with the same w/cm ; the construction was performed by one individual to eliminate the possibility of differences in this construction technique. Three specimens were tested with the first set of thin-bed mortar; these specimens are designated as TB1. The three specimens constructed with the second batch of thin-bed mortar were designated as TB2. The test results for these specimens are presented in Table 6.1.6c. The final average of all the specimens was 63.9 psi (0.44 MPa) with a corresponding COV of 44 percent. A 5 percent lower fractile of these results was 18 psi (0.12 MPa).

In some failed specimens, crushing of the material due to compression of a piece of the inner part of the legs was observed, and was attributed to the eccentricity of the reactions acting on the bottom of the specimens. Flexural cracks were also found on the outer side of the legs resulting from the eccentricity of reacting forces and tensile stresses due to Poisson effects. The eccentric reaction might produce local moments at the joint surfaces, decreasing the tested shear

Table 6.1.6b—Summary of results from direct-shear tests (thin-bed mortar only)

Specimen	Test age, days	Maximum load, kip (kN)	Maximum stress, psi (MPa)
DS-TB-1	20	26.2 (117)	112.8 (0.78)
DS-TB-2	20	7.9 (35)	34.0 (0.23)
DS-TB-3	20	18.6 (83)	79.8 (0.55)
DS-TB-4	21	17.5 (78)	75.1 (0.52)
DS-TB-5	21	13.5 (60)	58.1 (0.4)
		Average	72 (0.5)
		COV, %	40

Table 6.1.6c—Summary of results from additional direct-shear tests (thin-bed mortar only)

Specimen	Test age, days	Maximum load, kip (kN)	Maximum stress, psi (MPa)
DS-TB1-1	13	10.9 (49)	46.3 (0.3)
DS-TB1-2	13	19.3 (86)	81.9 (0.6)
DS-TB1-3	13	6.3 (28)	26.7 (0.2)
DS-TB2-1	14	6.9 (31)	29.1 (0.2)
DS-TB2-2	14	22.9 (102)	97.2 (0.7)
DS-TB2-3	14	14.5 (65)	61.4 (0.4)

strength. Because of this, the results proposed by Tanner (2003) may underestimate the shear strength of thin-bed mortar joints. Using the probable compressive strengths of these specimens, their shear strengths predicted using the linear regression from tests by Snow (1999) and Dembowski (2001) would have been between 80 and 100 psi (550 and 670 kPa). Until additional data are available, a lower-bound design strength of 18 psi (120 kPa) is recommended.

The requirement that the shear-bond behavior of AAC be governed by the material itself, rather than the mortar, is crucial to the development of design equations, testing requirements, and maximum limitations on the spacing of prescriptive reinforcement. It is essential to verify that with currently specified mortars, shear bond strength is governed by the AAC material itself.

6.1.7 Shear bond between AAC and grout—In an AAC structure, floor slabs are commonly composed of panels, joined by grouted keys, and panels adjacent to a bond beam. Reasonably reliable predictive equations for shear bond between AAC and grout are necessary to check the capacity of those joints. Direct shear strength can be evaluated in several ways and two test setups are presented in Fig. 6.1.7a and 6.1.7b. The first study tested traditional shear specimens and the second study evaluated grouted core strength. Another case of shear transfer is related to pullout of AAC cores in field-reinforced panels.

A direct shear specimen used to determine the shear capacity between grout and AAC is shown in Fig. 6.1.7a. Full blocks can be used for convenience. Three specimens were tested, and the average shear stress for each joint is 58 psi (400 kPa) with a COV of 22 percent (Tanner 2003).

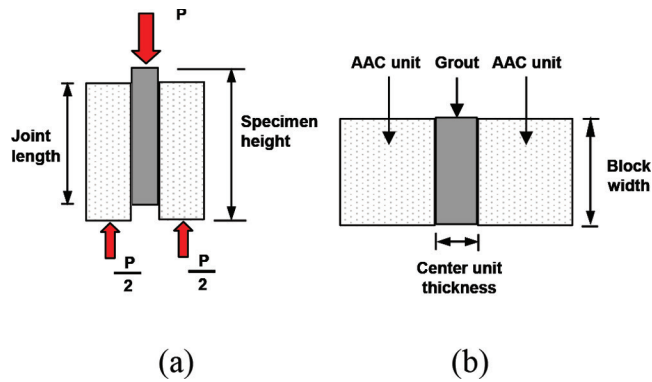


Fig. 6.1.7a—Direct-shear specimens: (a) elevation view; and (b) plan view.

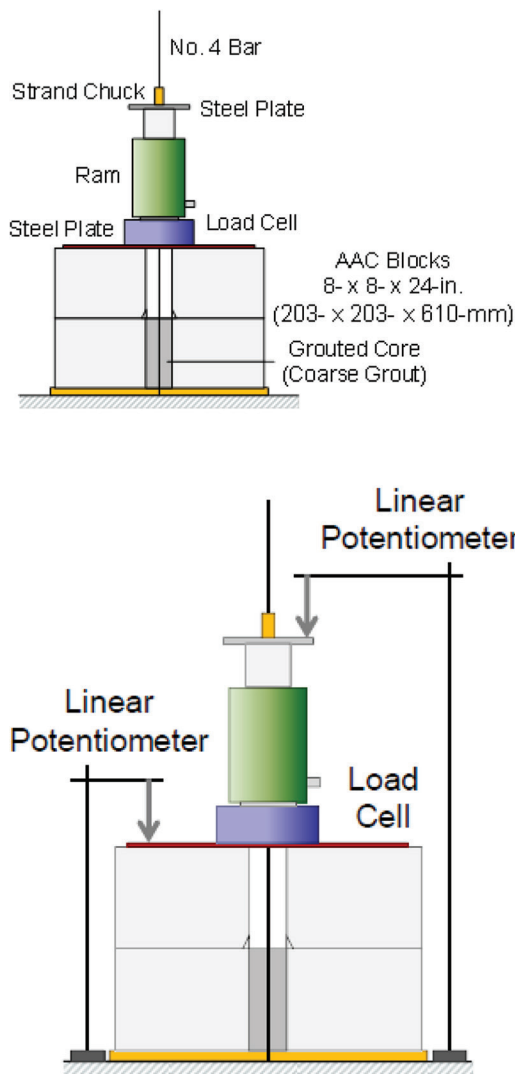


Fig. 6.1.7b—Details of interface shear tests by Forero Henao et al. (2012).

lower due to horizontal forces generated at the bottom of the AAC blocks. In turn, these forces generated flexural cracks on the outside of the AAC blocks.

Forero Henao et al. (2012) reported on the results of 18 pullout specimens using AAC blocks and coarse grout. Two full blocks were used with 4 in. (100 mm) cores in the center.

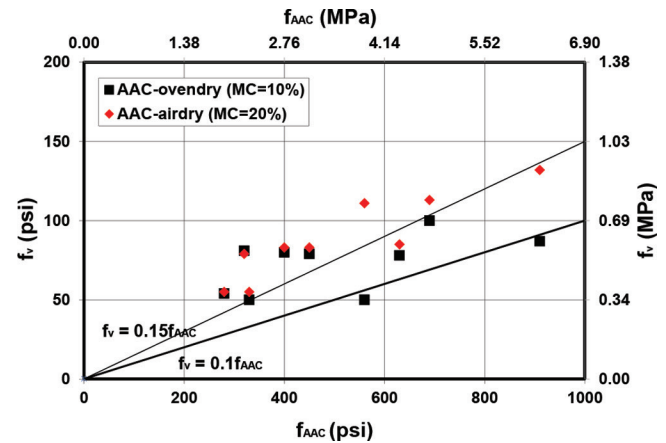


Fig. 6.1.8—Relationship between direct shear strength (f_v) and compressive strength (f_{AAC}) (Argudo 2003; Dembowski 2001; Snow 1999).

A No. 4 (5.1 mm) reinforcing bar was grouted into the bottom block. The upper block served as a reaction frame, as illustrated in Fig. 6.1.7b. The average shear stress was 74.4 psi (513 kPa) with a COV of 15 percent. A 5 percent lower fractile and 90 percent confidence interval results in a shear transfer of 49.9 psi (344 kPa). Based on these results, a reasonable value for interface shear transfer is 50 psi (345 kPa). This value is lower than the average test value of the smaller scale study mentioned previously.

6.1.8 Direct shear strength of AAC—Direct shear tests were conducted on plain AAC specimens. These specimens exhibited essentially zero moment and high shear (Dembowski 2001; Snow 1999). One group of specimens was oven-dried to a moisture content ranging from 8 to 12 percent (denoted as AAC oven-dry). A second group of specimens was air-dried to a moisture content ranging from 15 to 25 percent (AAC air-dry).

The relationship between direct shear strength f_v and compressive strength f_{AAC} is shown in Fig. 6.1.8, in which two sets of specimens ($f_{AAC} = 560$ and 910 psi [3.86 and 6.27 MPa]) show a significant decrease in shear strength from air-dried to oven-dried specimens. Because those points may be influenced by microcracking, it is debatable whether they should be included in the data analysis. For reasons of conservatism, they are included in the lower line denoted by Eq. (6.1.8). This is the recommended equation to estimate the nominal direct shear capacity of AAC. It provides results slightly above the 5 percent lower fractile of the complete set of data (Argudo 2003).

$$f_v = 0.1f_{AAC} \quad (6.1.8)$$

where f_{AAC} and f_v are in psi.

6.1.9 Coefficient of friction of AAC—The coefficient of friction between AAC and ASTM C270 Type S leveling mortar was determined based on sliding observed at four different times during the testing of Shear Wall Specimen 1 at UT Austin (Tanner 2003). Because no vertical reinforcement was present in the wall, the sliding behavior came from axial load alone. The coefficient of friction was

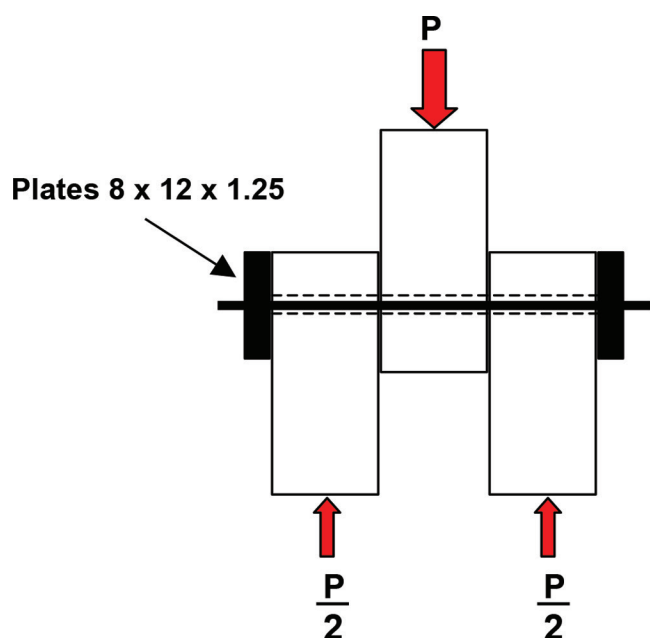


Fig. 6.1.9—Direct shear specimen to determine coefficient of friction for AAC (Tanner 2003).

Table 6.1.9—Results for coefficient of friction between two pieces of AAC (Tanner 2003)

Specimen	Maximum vertical load, kip (kN)	Measured clamping force, kip (kN)	Coefficient of friction
1 – 5	7.7 (34)	4.8 (21)	0.80
2 – 5	9.3 (41)	5.3 (24)	0.87
3 – 5	8.8 (39)	4.8 (21)	0.93
1 – 10	13.7 (61)	9.2 (41)	0.75
2 – 10	20.3 (90)	10.6 (47)	0.96
3 – 10	17.1 (76)	10.3 (46)	0.83
1 – 15	24.4 (109)	14.1 (63)	0.86
2 – 15	27.9 (124)	16.1 (72)	0.87
3 – 15	21.5 (96)	14.0 (62)	0.77

calculated by dividing the lateral load by the corresponding applied axial load at each time sliding occurred, as shown in Eq. (6.1.9a). The average of those four values is 1.0, with a COV of 4 percent.

$$V_{sliding} = \mu N \quad (6.1.9a)$$

Because the coefficient of friction between two pieces of AAC may differ from that between AAC and leveling mortar, additional direct-shear tests were performed using unmortared AAC units clamped together with threaded rods (Fig. 6.1.9) running through the centerline of a 3 in. (76 mm) core in each AAC modular block. Three specimens were constructed and each was subjected to three levels of applied clamping force: 5, 10, and 15 kip (22, 44, and 67 kN). The test results are presented in Table 6.1.9. The coefficient of friction is determined by Eq. (6.1.9b), where N is the clamping force. The average coefficient of friction is 0.8,

with a COV of 8 percent. A 10 percent lower fractile is 0.76, and the proposed design value for coefficient of friction between two pieces of AAC is 0.75 (Tanner 2003).

$$\mu = \frac{P}{2N} \quad (6.1.9b)$$

6.1.10 Freezing-and-thawing resistance of AAC—Exterior AAC should be treated coated with a breathable water-resistant coating.

6.2—Bond strength between factory-installed wire reinforcement and AAC

Testing performed on beams and shear walls constructed of factory-produced reinforced AAC panels indicate a potential anchorage failure governed by crushing of AAC surrounding welded-wire reinforcement. Beam and shear wall test results are evaluated in the following sections, followed by proposed design provisions.

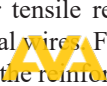
6.2.1 Anchorage behavior of reinforced AAC panels—A series of out-of-plane bending tests on precast AAC panels was performed by Fouad and Dembowski (2005b) and Snow (1999), as illustrated in Fig. 6.2.1a. Of the 12 reinforced floor panels tested, three were reported to fail in the anchorage zone. In this section, this behavior is discussed, a predictive model for flexure is proposed, and the implications of that model are presented.

For each panel, the flexural lever arm is taken as the distance between the layers of welded-wire mesh (Fig. 6.2.1b). Due to the low compressive strength of AAC, this arm is slightly shorter than the traditional value of $0.9d$ often assumed in design of conventional reinforced concrete.

As shown in Fig. 6.2.1c, the tensile force in the tensile reinforcement is limited by the bearing capacity of the AAC under the cross wires. The equations predicting the demand in the tensile layer of steel and the resistance based on bearing of the cross wires are presented in Eq. (6.2.1a) and (6.2.1b), respectively. In the capacity equation, n_{cross} refers to the number of cross wires and d_{cross} refers to their diameter. Possible additional frictional resistance at the support reaction was calculated as the product of the support reaction times a coefficient of friction of 0.5, selected based on engineering judgment. In Table 6.2.1, this resistance is compared to the capacity. The length of each panel in feet (meters) is the final value in each specimen name. In eight cases, failure is predicted but only three failures were observed. Results are summarized in Table 6.2.1.

$$T_{demand} = VL/(4 \cdot \text{arm}) \quad (6.2.1a)$$

$$T_{capacity} = n_{cross} d_{cross} l_{cross} f_{AAC} \quad (6.2.1b)$$

The implications of these predictions are important. Equation (6.2.1c) relates the average bond stress (u_{ave}) to the change in the force in the wire (ΔT). The term d_{long} is the diameter of the longitudinal or tensile reinforcement, and n_{long} is the number of longitudinal wires. For the case of Fig. 6.2.1b, ΔT is the tensile force in the reinforcement.  **Documents**
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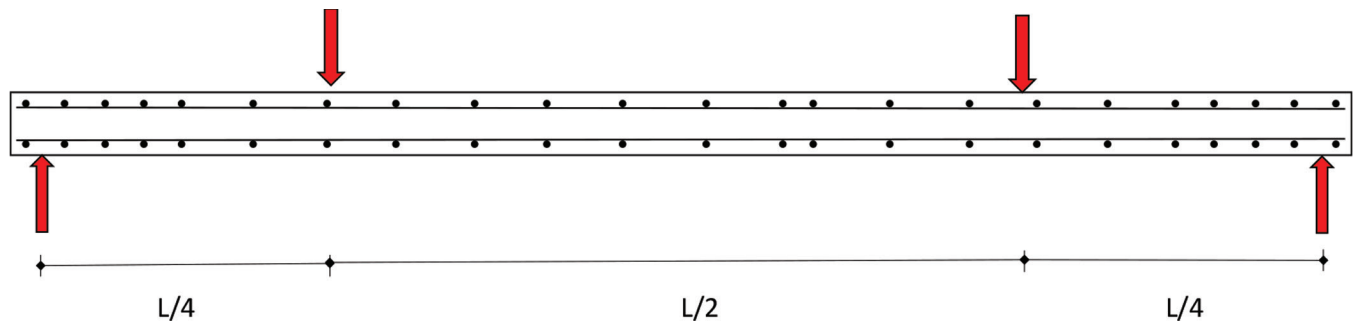


Fig. 6.2.1a—Reinforced AAC panel subjected to flexural loads (Dembowski 2001; Snow 1999).

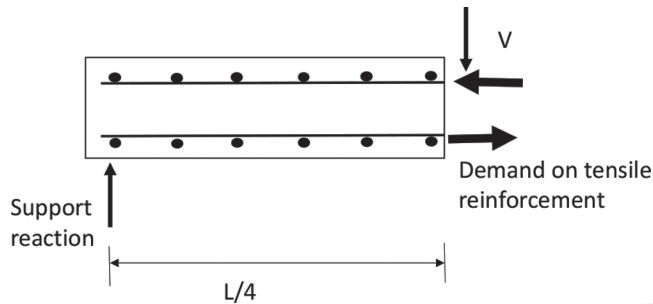


Fig. 6.2.1b—Free-body diagram of panel section (Tanner 2003; Argudo 2003).

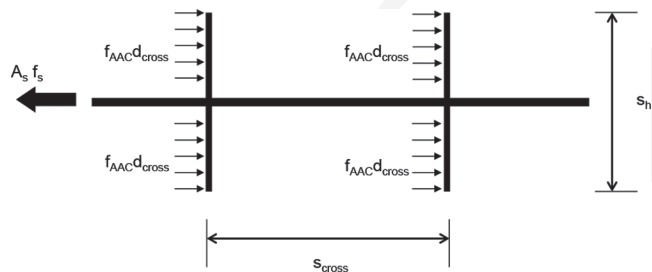


Fig. 6.2.1c—Resistance mechanism based on cross wires functioning in bearing (Tanner 2003; Argudo 2003).

and dx is the length $L/4$. By substituting those values into Eq. (6.2.1c), a relationship between shear and average bond stress can be expressed in terms of shear (Eq. (6.2.1d)). If the average bond stress is proportional to shear, then so is the bearing stress of the cross wires on the AAC.

$$\frac{\Delta T}{dx} = u_{ave} d_{long} \pi \quad (6.2.1c)$$

$$\frac{V}{n_{long} \pi d_{long} \text{arm}} = u_{ave} \quad (6.2.1d)$$

For a case of uniform shear, as in the reinforced panel tests subject to quarter-point bending, the bearing stress on AAC is uniform. The panel capacity associated with this limit state can be predicted based on local crushing of the AAC under the cross wires. The maximum force in the tensile reinforcement will cause local crushing of the cross wire bearing against the AAC at the full compressive strength. As load is increased, cross wires not fully bearing on the AAC will

begin to exert more bearing stress. By such redistribution, the load can increase until all cross wires participate evenly.

In the case of nonuniform shear in a section, the cross wires near the support will be most highly stressed first, as the shear is maximum there. As the AAC crushes in bearing, the bearing stress on the adjacent cross wires will increase. For a given problem, if the bond stress is uniform, the required tensile force in the reinforcement can be determined based on equilibrium, and the available resistance can be predicted to avoid anchorage failure. This model was confirmed by Fadaifard (2008) using panels subjected to in-plane and out-of-plane gravity loads.

6.2.2 Anchorage behavior of tensile reinforcement for shear walls—Three shear-dominated wall specimens, tested by Varela (2003) and Tanner (2003), were made of horizontal panels with welded-wire reinforcement. The response of those walls gives additional information on the average bond strength associated with bearing of the cross wires on the AAC. The difference between their maximum shear capacity (V_n) and the base shear at web-shear cracking (V_{AAC}) provides an estimate of the contribution of the welded-wire reinforcement. The estimate depends on the following assumptions:

- All cross wires in a critical length of panel bear on the AAC with a maximum stress equal to the compressive strength of the AAC
- All cross wires that cross the web-shear crack participate equally

Although the second assumption is not consistent with testing of masonry shear walls, it was useful for estimating bond strengths. As discussed in 4.2.3, cross wires are not effective under reversed cyclic loads. While this is relevant to AAC shear walls, it is not relevant to factory-reinforced AAC elements typically used as beams. Beams are typically used as lintels to support gravity loads and typically not subjected to reversed cyclic loads. The following discussion pertains to this application.

The transfer of shear across the web-shear crack is unknown. The average bond strength can be estimated for two limiting conditions: 1) all V_{AAC} is resisted by shear across the crack; and 2) none of V_{AAC} is resisted by shear across the crack. Based on the latter evaluation, and information from reinforced panels presented in Table 6.2.2, calculated bond strengths range from 72 to 128 psi (496 to 882 kPa), which

Table 6.2.2—Calculated range of bond strengths for welded-wire fabric based on shear walls tests (Tanner 2003; Varela 2003)

Specimen	Maximum capacity V_n , kip (kN)	Observed V_{AAC} , kip (kN)	u with V_{AAC} , ksi (MPa)	u without V_{AAC} , psi (MPa)
1	168 (745)	164 (730)	3 (21)	86,200 (594)
4	126 (562)	82 (366)	15 (103)	2300 (16)
9	42 (188)	42 (188)	0.2 (1)	3000 (21)

are lower than the range of 2300 to 86,200 psi (15.9 to 594 MPa) calculated for shear wall specimens neglecting V_{AAC} .

6.2.3 Design proposal for anchorage capacity—Equations (6.2.1a) and (6.2.1b) were proposed for predicting the capacity, as governed by anchorage failure, for the reinforced panels of 6.2.1 (Argudo 2003). In the tests reported in Table 6.2.1, the shear force and the variation of the tensile force (dT_{demand}/dx) were uniform within the shear span ($L/4$), making it possible to calculate the minimum number of equally spaced cross wires (n_{cross}) required to avoid anchorage (bond) failure.

Equations (6.2.1a) and (6.2.1b) are generalized and rewritten as Eq. (6.2.3a) and (6.2.3b) for a beam subjected to distributed loads and variable shear along its length.

$$\frac{dT_{demand}(x)}{dx} = \frac{dM(x)}{dx} \cdot \frac{1}{\text{arm}} = \frac{V(x)}{\text{arm}} \quad (6.2.3a)$$

$$\frac{dT_{capacity}(x)}{dx} = \frac{dn_{cross}(x)}{dx} \cdot d_{cross} \cdot l_{cross} \cdot f_{AAC} \quad (6.2.3b)$$

Making the demand Eq. (6.2.3a) equal to the capacity Eq. (6.2.3b) results in Eq. (6.2.3c), which gives the number of cross wires per unit length required to resist the tensile force in the longitudinal reinforcement, as a function of the shear in the panel and its cross-sectional geometry.

$$\frac{dn_{cross}(x)}{dx} = \frac{V(x)}{\text{arm} \times d_{cross} \times l_{cross} \times f_{AAC}} \quad (6.2.3c)$$

For design purposes, it is not recommended to compute the number of cross wires (n_{cross}) as a variable of the length of the beam. Instead, the minimum cross-wire spacing (s_{min}) or, alternatively, the minimum number of equally spaced cross wires ($n_{cross,min}$) corresponding to the maximum shear force (V_{max}) in a certain shear span can be computed by Eq. (6.2.3d) and (6.2.3e).

For a simply supported floor panel with uniformly distributed load, the variation of the tensile force (dT_{demand}/dx) becomes critical at the supports and at a distance $L/6$ from them, requiring the longitudinal reinforcement to be anchored by the cross wires. Substituting $a = L/6$ and taking the lever arm as $0.85d$, Eq. (6.2.3e) can be rewritten in terms of a minimum number of cross wires as Eq. (6.2.3f).

$$s_{min} = \frac{V_{max}}{0.85d \cdot d_{cross} \cdot l_{cross} \cdot f_{AAC}} \quad (6.2.3d)$$

$$n_{cross,min} = \frac{V_{max}}{0.85d \cdot d_{cross} \cdot l_{cross} \cdot f_{AAC}} \cdot a \quad (6.2.3e)$$

$$n_{cross,min} = \frac{V_{max} \cdot L}{5.1 \cdot d \cdot d_{cross} \cdot l_{cross} \cdot f_{AAC}},$$

to a distance $L/6$ from supports (6.2.3f)

For the intermediate zone between $L/6$ and $L/2$ from each support, a minimum number of cross wires needs to be specified to permit redistribution of bearing stresses along the length of the panel. Therefore, in those zones, it is recommended to use the same number of cross wires at twice the spacing calculated for the anchorage zones ($L/6$ from each support).

Finally, a strength-reduction factor ($\phi = 0.75$) should be introduced to modify Eq. (6.2.3d) through (6.2.3f) for design (Eq. (6.2.3g)).

$$V_{max} = \frac{V_u}{\phi} = \frac{V_u}{0.75} \quad (6.2.3g)$$

6.3—Flexural design of AAC beams

6.3.1 Nominal flexural capacity (M_n)—Nominal flexural capacity (M_n) can be calculated using conventional assumptions of plane sections, strain compatibility, stress-strain relationships, and equilibrium. The compressive zone is determined based on a linear stress-strain relationship, using a maximum useful compressive strain in the AAC of 0.003, and an equivalent rectangular stress block with a height of $0.85f_{AAC}'$, and a depth of $\beta_1 c$, where $\beta_1 = 0.67$.

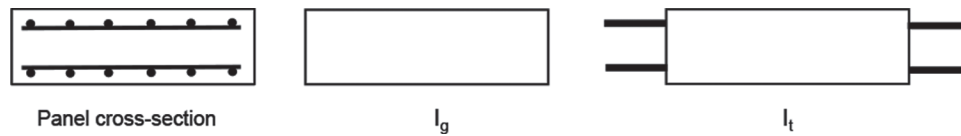
Using these relationships, Argudo (2003) found very good agreement between experimentally derived and analytically determined nominal moment capacities (M_n) for reinforced AAC panels failing predominantly in flexure (Panels 1-2 YF 6-24-12.5 A and 1-2 YF 8-24-16.5 A).

The design flexural capacity (ϕM_n) is obtained by multiplying M_n by a strength-reduction factor $\phi = 0.9$ when strains in the steel exceed 0.005.

6.3.2 Minimum flexural reinforcement ratio—Selection of the minimum reinforcement quantity, A_{smin} , for AAC flexural elements is based on the objective that the nominal flexural capacity M_n be equal to at least twice the cracking moment M_{cr} . For a rectangular section, M_n and M_{cr} are approximately defined by Eq. (6.3.2a) and (6.3.2b), respectively. Assuming that $I_t = 1.5I_g$ and $\left(d - \frac{a}{2}\right) = 0.7h$, the following equations

are derived. Illustrations of variables and dimensions are provided in Fig. 6.3.2.

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 0.7h A_s f_y \quad (6.3.2a)$$

Fig. 6.3.2—Illustrations of cross sections I_g and I_t .

$$M_{cr} = f_r S = f_r \left(\frac{I_t}{y_t} \right) = 4.8 \sqrt{f'_{ACC}} \left(1.5 \frac{b_w h^3}{h} \right) = 1.2 \sqrt{f'_{ACC}} b_w h^2 \quad (6.3.2b)$$

$$2M_{cr} = 2.4 \sqrt{f'_{ACC}} b_w h^2 \quad (6.3.2c)$$

Equating Eq. (6.3.2a) and Eq. (6.3.2c), assuming that $h = 1.15d$ and solving for A_s , the minimum reinforcement quantity, $A_{s,min}$ is found and given by Eq. (6.3.2e). For convenience, the leading coefficient is rounded up from 3.94 to 4 in Eq. (6.3.2f).

$$0.7hA_s f_y = 2.4 \sqrt{f'_{ACC}} b_w h^2 \quad (6.3.2d)$$

$$A_s = 3.43 \frac{\sqrt{f'_{ACC}}}{f_y} b_w h = 3.43 \frac{\sqrt{f'_{ACC}}}{f_y} b_w (1.15d) = \frac{3.94 \sqrt{f'_{ACC}} b_w d}{f_y} \quad (6.3.2e)$$

$$A_{s,min} = \frac{4 \sqrt{f'_{ACC}} b_w d}{f_y} \quad (6.3.2f)$$

where f'_{ACC} and f_y are in psi.

6.3.3 Flexural design of tension- and compression-controlled AAC elements under ACI 318—Following 7.3.3 or 9.3.3 of ACI 318-14, beams or one-way floor systems cannot have maximum tensile strains less than 0.004, and this requirement should be checked using the specified yield strength of flexural reinforcement, which is often $f_y = 80$ ksi (552 MPa) for smooth wire. Designers need to use appropriate confining reinforcement to avoid buckling of the welded-wire fabric. This is not standard in factory-reinforced AAC panels.

Chapter 21 of ACI 318-14 should be applied using a strength reduction factor of $\phi = 0.65$ for compression-controlled sections, $\phi = 0.9$ for tension-controlled sections, and ϕ equal to a value linearly interpolated between 0.65 and 0.9 when the strain in the extreme tensile reinforcement (ϵ_t) is between 0.002 and 0.005. Because of the limitation of 9.3.3, the strength reduction factor will never be less than 0.82 for beam elements.

6.4—Control of deflections

Experimentally measured deflections are to be consistently higher (by a factor of 1.25 to 1.8) than deflections calculated using the effective moment of inertia (I_e) given in 24.2.3.5 of ACI 318-14 (Fouad and Dembowski 2005b; Dembowski 2001; Snow 1999; Argudo 2003). Because these equations are implicitly linked to Table 7.3.1.1 of ACI 318-14, both this table and the deflection-calculation equations need to be modified for use with AAC elements.

The minimum thickness requirements of Table 7.3.1.1 of ACI 318-14 can be applied to reinforced AAC elements if the following constraints are applied:

- Section 7.3.1.1.1 of ACI 318-14 should be applied. It specifies that for structural lightweight concrete having unit weight in the range 90 to 120 lb/ft³ (1440 to 1920 kg/m³), the minimum thickness shall be multiplied by $(1.65 - 0.005w_c)$, where w_c is the unit weight in lb/ft³ (kg/m³). The footnote is applicable, because AAC is comparable to lightweight concrete in terms of its flexural stiffness. For the reinforced AAC panels tested by Fouad and Dembowski (2005b), the corresponding factor from ACI 318-14 Section 7.3.1.1.1 is 1.47, making the ratios of calculated to observed deflections all comfortably above unity.
- ACI 318-14 Section 7.3.1.1.2 requires that for f_y other than 60,000 psi, calculated deflection values be multiplied by $(0.4 + f_y/100,000)$.

The deflection-calculation provisions of 7.3.1.1 and 24.2.4.1 of ACI 318-14 can be applied to reinforced AAC elements, with the following modifications:

- Short-term deflections should be calculated using an effective flexural stiffness (EI_e) corresponding to the unfactored moment (M_a). The effective flexural stiffness (EI_e) should be obtained by linear interpolation between the cracking point (M_{cr} , ϕ_{cr}) and the yielding points (M_y , ϕ_y) on a bilinear moment-curvature diagram. This procedure is recommended by RILEM Technical Committees 78-MCA and 51-ALC (1993). The short-term deflections so obtained should then be multiplied by the factor from ACI 318-14 Section 7.3.1.1.1. When this approach is used, calculated deflections are 0.9 to 1.2 times the experimentally observed values.

- For calculation of additional long-term deflections resulting from creep and shrinkage of AAC flexural members, Section 24.2.4.1 of ACI 318-14 is not applicable because the reinforcement ratio for compressive reinforcement (ρ') is generally much smaller for AAC than for reinforced concrete. To calculate total deflections, including long-term deflections, a simplified approach based on RILEM Technical Committees 78-MCA and 51-ALC (1993) can be adopted. Total deflections can be calculated using an effective modulus of elasticity (E_{AAC}) equal to the modulus of elasticity (E_{AC}) divided by 1.5.

6.5—Shear design of AAC beam elements

6.5.1 Shear capacity of AAC (V_{AAC})—For reinforced concrete beams without shear reinforcement, the mean diagonal tensile strength f_t can be taken equal to $6\sqrt{f'_c}$ ($0.5\sqrt{f'_c}$). Table 22.5.5.1 of ACI 318-11 uses a nominal unit strength in shear that decreases from $3.5\sqrt{f'_c}$ ($0.29\sqrt{f'_c}$) for low span-to-depth ratios, to $1.9\sqrt{f'_c}$ ($0.16\sqrt{f'_c}$) for high ratios. For simplicity, Eq. (22.5.5.1) of ACI 318-14 can be used with a corresponding unit strength of $2\sqrt{f'_c}$ ($0.17\sqrt{f'_c}$). That unit strength is one-third of the mean diagonal tensile strength.

On that basis, Eq. (22.5.5.1) can be rewritten in terms of the splitting tensile strength f_t for use with AAC elements. The corresponding shear capacity (V_{AAC}) is given by Eq. (6.5.1a) for members subjected to shear and flexure only, and by Eq. (6.5.1b) for members subjected to axial compression as well (Argudo 2003).

$$V_{AAC} = \frac{f_t}{3} b_w d \quad (6.5.1a)$$

$$V_{AAC} = \frac{f_t}{3} \left(1 + \frac{N_u}{2000A_g} \right) b_w d \quad (6.5.1b)$$

Substituting Eq. (6.1.3.1) into Eq. (6.5.1a) and (6.5.1b), Eq. (6.5.1c) and (6.5.1d) are obtained as follows

$$V_{AAC} = 0.8\sqrt{f_{AAC}} b_w d \quad (\text{in.-lb}) \quad (6.5.1c)$$

$$V_{AAC} = 0.066\sqrt{f_{AAC}} b_w d \quad (\text{SI})$$

where V_{AAC} is in lb and f_{AAC} is in psi.

$$V_{AAC} = 0.8\sqrt{f_{AAC}} \left(1 + \frac{N_u}{2000A_g} \right) b_w d \quad (\text{in.-lb}) \quad (6.5.1d)$$

$$V_{AAC} = 0.066\sqrt{f_{AAC}} \left(1 + \frac{N_u}{2000A_g} \right) b_w d \quad (\text{SI})$$

where V_{AAC} is in lb and f_{AAC} is in psi.

In addition, the design compressive strength of AAC, f'_{AAC} , is used in the final design equations, Eq. (6.5.1c) and (6.5.1d).

6.5.2 Shear strength provided by shear reinforcement (V_s)—Shear strength provided by shear reinforcement V_s can be calculated using Section 22.5.10.5.3 of ACI 318-14, with the following modifications based on Tanner (2003):

- Vertical or inclined wires designed to provide shear strength need to be welded to the longitudinal reinforcement.
- The maximum usable shear strength provided by each shear reinforcing bar V_{sb} is limited by the bearing capacity of the AAC on the longitudinal reinforcement. Equation (6.5.2) should be verified.

$$V_{sb} \leq d_{long} \cdot s \cdot f_{AAC} \quad (6.5.2)$$

where d_{long} is the diameter of longitudinal reinforcement; s is the shear reinforcement spacing along the beam axis; and f_{AAC} is the compressive strength of AAC.

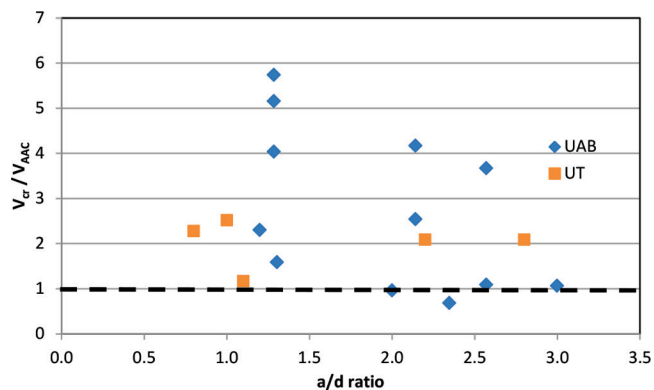


Fig. 6.5.4—Observed versus predicted capacities of beam tests as governed by shear cracking (Fouad et al. 2002; Yousef 2009).

6.5.3 Design shear capacity (ϕV_n)—The nominal shear capacity V_n is equal to the sum of the nominal shear capacity of AAC (V_{AAC}) and the shear strength provided by the shear reinforcement V_s . The design shear capacity ϕV_n is obtained by multiplying the nominal shear capacity V_n by a strength reduction factor $\phi = 0.75$.

6.5.4 Predicted capacities of reinforced AAC floor panels—A suite of reinforced panels was tested in flexure by Fouad and Dembowski (2005b), as illustrated in Fig. 6.5.4. Shear capacities are calculated using Eq. (6.5.2), flexural capacities are calculated as explained in 6.3.1, and anchorage capacities are computed as the summation of the resistance from bearing and the resistance from friction, as in Table 6.2.1. Capacity/demand ratios can be used to predict governing failure modes. Failures in those panels were predicted to occur by a combination of shear and anchorage failure. Many panels in Table 6.5.4a show capacity/demand ratios close to or less than 1.0 for shear and for anchorage. In some cases, reported experimental observations agreed with these predictions. In other cases, the nature of the observed failure mode was not specified (Dembowski 2001; Snow 1999).

The same procedure was followed for the panels tested by Construction Technology Laboratories (1999). Results are presented in Table 6.5.4b.

Beam tests conducted by Fouad et al. (2002) and Yousef (2009) report the applied load at cracking. This permits an analysis that compares the cracking load to shear strength provided by AAC as defined in Eq. (6.5.1c). Capacity/demand ratios for these beams are presented in Fig. 6.5.4. Square markers represent the average of five or more identical beams and diamond markers represent the average of two identical beams. Results are tabulated in Table 6.5.4c. Both sets of data indicate that Eq. (6.5.1c) is conservative.

6.6—Design of AAC shear walls

A suite of 14 AAC shear wall specimens with aspect ratios (height of the point of load application divided by the plan length) from 0.6 to 3 was tested (Tanner et al. 2005a,b; Varela et al. 2006). The behavior of each shear wall may be shear- or flexure-dominated. The shear-dominated specimens were heavily reinforced in flexure using external reinforcement.

Table 6.5.4a—Summary of predicted and observed modes of failure for floor panels tested by Fouad and Dembowski (2005b) and Snow (1999)

Specimen	Shear		Flexure		Anchorage		Reported failure mode (UAB)	Predicted failure mode (UT Austin)
	V_{AAC} observed, kip (kN)	V_{AAC} predicted, kip (kN)	M_n observed, kip-in. (kN-m)	M_n predicted, kip-in. (kN-m)	T demand, kip (kN)	T capacity, kip (kN)		
1 YF 6-24-12.5 A	2.1 (9.2)	2.2 (9.8)	80.7 (9.1)	80.9 (9.1)	16.1 (71.6)	17.8 (79.1)	Not specified	Flexure
2 YF 6-24-12.5 A	2.1 (9.4)	2.2 (9.8)	83.1 (9.4)	80.9 (9.1)	16.6 (73.8)	17.9 (79.6)		
1 YF 8-24-16.5 A	2.4 (10.5)	3.0 (13.3)	119.4 (13.5)	119.1 (13.4)	16.1 (71.6)	18.0 (80.0)		Flexure
2 YF 8-24-16.5 A	2.4 (10.6)	3.0 (13.3)	119.4 (13.5)	119.1 (13.4)	16.1 (71.6)	18.0 (80.0)		
1 YF 6-24-12.5 B	2.4 (10.7)	2.2 (9.8)	93.6 (10.6)	116.8 (13.2)	19.0 (84.5)	18.2 (80.9)		Shear/Anchorage
2 YF 6-24-12.5 B	2.4 (10.6)	2.2 (9.8)	93.3 (10.5)	116.8 (13.2)	18.9 (84.0)	18.2 (80.9)		
1 YF 8-24-16.5 B	3.2 (14.0)	3.0 (13.3)	158.3 (17.9)	174.1 (19.7)	22.1 (98.2)	18.8 (83.6)		Shear/Anchorage
2 YF 8-24-16.5 B	3.4 (15.2)	3.0 (13.3)	171.7 (19.4)	174.1 (19.7)	24.2 (108)	19.1 (84.9)		Shear/Anchorage
1 YF 6-24-12.5 C	3.4 (15.2)	3.0 (13.3)	171.7 (19.4)	174.1 (19.7)	24.2 (108)	19.1 (84.9)	Anchorage	Shear/Anchorage
2 YF 6-24-12.5 C	3.1 (13.7)	2.2 (9.8)	118.9 (13.4)	178.3 (20.1)	24.6 (109)	18.9 (84.1)		
1 YF 8-24-16.5 C	3.2 (14.2)	2.2 (9.8)	123.1 (13.9)	178.3 (20.1)	25.5 (113.3)	19.0 (84.5)	Anchorage	Shear/Anchorage
2 YF 8-24-16.5 C	3.1 (13.7)	2.2 (9.8)	118.9 (13.4)	178.3 (20.1)	24.6 (109.3)	18.9 (84.0)	Not specified	

Table 6.5.4b—Summary of predicted and observed modes of failure in ACCOA panels (Construction Technology Laboratories, Inc. 1999; Argudo 2003)

Specimen	Shear		Flexure		Anchorage		Reported failure mode (CTL)	Predicted failure mode (UT Austin)
	V_{AAC} observed, kip (kN)	V_{AAC} predicted, kip (kN)	M_n observed, kip-in. (kN-m)	M_n predicted, kip-in. (kN-m)	T demand, kip (kN)	T capacity, kip (kN)		
S1-8-24-15	3.84 (17.1)	3.24 (14.4)	159 (17.9)	153 (17.3)	25.4 (113)	19.8 (88)	Shear	Flexure/Shear/Anchorage
S2-8-24-15	3.62 (16.1)	3.24 (14.4)	150 (16.9)	153 (17.3)	24.0 (107)	19.6 (87)		Shear/Anchorage
S3-8-24-15	3.18 (14.1)	3.24 (14.4)	132 (14.8)	153 (17.3)	21.0 (93)	19.1 (85)		Anchorage
WT1-8-24-14	3.08 (13.7)	3.24 (14.4)	137 (15.4)	153 (17.3)	21.9 (97)	19.0 (85)		Shear/Anchorage
WT2-8-24-14	3.31 (14.7)	3.24 (14.4)	147 (16.6)	153 (17.3)	23.5 (105)	19.3 (86)		
WT3-8-24-14	2.57 (11.4)	3.24 (14.4)	114 (12.9)	153 (17.3)	18.2 (18.1)	18.5 (82)		Shear

Table 6.5.4c—Summary of predicted and observed capacities for beam tests (Fouad et al. 2002; Yousef 2009)

Test location	Specimen	a/d	$V_{cr} = V_{AAC}$ observed, kip (kN)	V_{AAC} predicted, kip (kN)	Observed V_{AAC} /predicted V_{AAC}
UAB	HF 8-8-3	1.29	5.46 (24.30)	1.06 (4.72)	5.15
	HF 8-8-5	2.14	4.42 (19.66)	1.06 (4.72)	4.17
	HF 8-8-6	2.57	3.89 (17.29)	1.06 (4.72)	3.66
	HF 8-8-7	1.29	7.60 (33.81)	1.33 (5.90)	5.73
	HF 10-8-3	2.14	3.36 (14.96)	1.33 (5.90)	2.54
	HF 10-8-5	3.00	1.41 (6.26)	1.33 (5.90)	1.06
	YF 8-8-3	1.29	4.67 (20.76)	1.16 (5.14)	4.03
	YF 8-8-6	2.57	1.25 (5.57)	1.16 (5.14)	1.08
	YF 8-16-6	1.20	5.69 (25.32)	2.48 (11.02)	2.30
	YF 8-24-10	2.00	2.38 (10.57)	2.48 (11.02)	0.96
	YF 8-24-10	1.30	6.01 (26.76)	3.80 (16.90)	1.58
	YF 8-24-10	2.35	2.58 (11.49)	3.80 (16.90)	0.68
UT	12-072	0.80	3.59 (15.98)	1.58 (7.03)	2.28
	12-096	1.10	1.84 (8.19)	1.58 (7.03)	1.17
	24-072	1.00	8.42 (37.47)	3.34 (14.86)	2.52
	24-096	2.20	6.96 (30.97)	3.34 (14.86)	2.09
	24-120	2.80	6.98 (31.06)	3.34 (14.86)	2.09