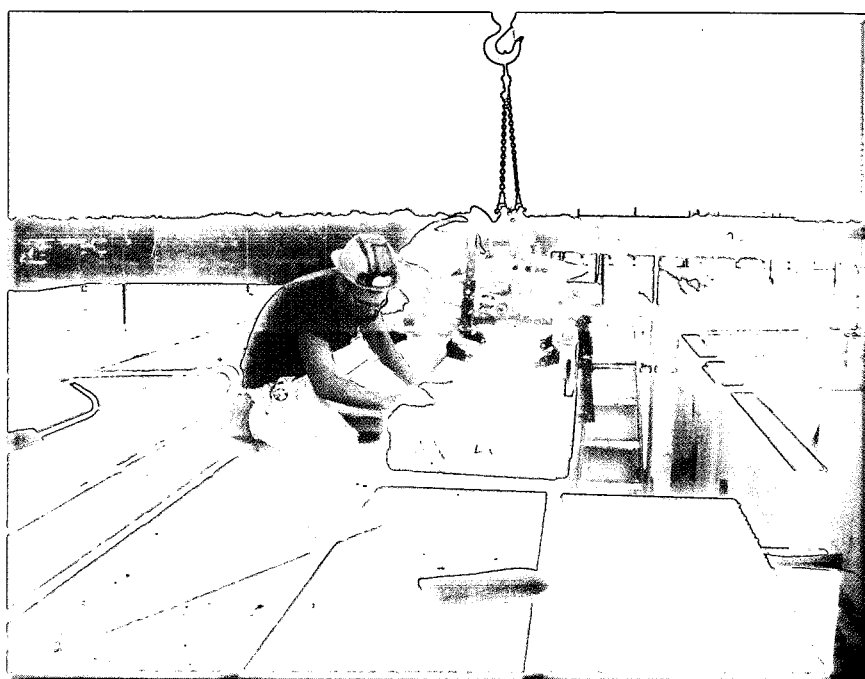


AUTOCLAVED AERATED CONCRETE— PROPERTIES AND STRUCTURAL DESIGN



Editor
Caijun Shi
Fouad H. Fouad



American Concrete Institute®
Advancing concrete knowledge

SP-226

Autoclaved Aerated Concrete—Properties and Structural Design

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PREFACE

Since its inception more than 80 years ago, autoclaved aerated concrete (AAC) has enjoyed a reputation for excellent thermal insulation, acoustic, and fire-resistance properties due to its low density and cellular structure. The production and use of AAC in the United States, however, did not start until the mid 1990s. To promote and encourage the use of AAC and other ultra-lightweight concrete, ACI Committee 523, Cellular Concrete, and ACI Committee 229, Controlled Low-Strength Materials, organized a technical session on “Controlled-Density/Controlled-Strength Concrete Materials” at the 2003 ACI Spring Convention in Vancouver, Canada, and a session on “Aerated Concrete – An Innovative Building Solution” at the 2003 ACI Fall Convention in Boston. Thirteen papers were presented at these two technical sessions covering a wide range of practical case studies and research projects on different types of ultra-lightweight concretes, with particular focus on AAC. These papers should be of interest to the practicing engineers, educators, and researchers in that they demonstrate the effective use of AAC as well as other types of ultra-lightweight concrete materials.

This special publication (SP) contains eight of the 13 papers presented at the session. Six of the papers deal with AAC and cover a wide variety of topics including material properties, structural design, seismic performance, and design examples. The other two papers address the acoustic and structural properties of foamed and/or aerated lightweight concretes cured at room temperature.

On behalf of ACI Committees 229 and 523, the editors would like to thank the authors and presenters for their contributions to the two technical sessions and this volume, the reviewers of the original manuscripts for their constructive comments and suggestions, and ACI staff for their great help in organizing the sessions and preparing this special publication. It is our hope that the success of these technical sessions and the publication of this SP will encourage and facilitate the use of AAC and other ultra-lightweight concrete materials in construction.

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Mechanical Properties of Plain AAC Material

by **F. H. Fouad and J. Dembowski**

Synopsis: Autoclaved aerated concrete (AAC) is a lightweight uniform cellular material, first developed in Sweden in 1929. Since that time, AAC building components have been widely used in Europe and other parts of the world. Until recently, however, AAC was relatively unknown to the United States precast construction market. Today, AAC is gaining rapid acceptance in the United States due primarily to increasing energy cost and environmental concerns.

Although AAC is a well-recognized building material in Europe, very little research work has been done on American-produced AAC components. The goal of the testing program was to further develop the database of the material properties and structural behavior of American-made AAC by testing plain AAC elements from three different manufacturers. Manufacturers and designers will be provided with information that will help to promote AAC as a reliable engineered construction material in the U.S. Tests performed on the plain AAC consisted of compressive strength, flexural tensile strength, shear strength, and modulus of elasticity.

Keywords: autoclaved aerated concrete; material properties; specifications; test methods

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INTRODUCTION

Autoclaved aerated concrete (AAC) is a lightweight uniform cellular material, first developed in Sweden in 1929, and based on a patented process by Johan Eriksson. However, the first major production plant was not constructed in the U.S. until 1996. Today, AAC is gaining rapid acceptance as a new building product in the U.S. as a result of the increasing importance placed on energy, since energy savings are realized both in the production process of AAC and in the thermal insulation properties of the AAC finished product [1]. The rising cost of lumber and increasing environmental concerns have also played a role in the interest surrounding AAC.

The AAC production process is very sensitive to the quality of the materials used in the concrete mix and their proportions. The raw materials consist of Portland cement, finely ground sand, and lime. In some cases, it is acceptable to replace the finely ground sand with fly ash. These materials are mixed with water and a small amount of aluminum powder and cast into a mold. Hydrogen gas, a product of the reaction between the cement hydration products and the aluminum powder, causes the material to rise in the mold, creating macroscopic air cells throughout the material. After 3 to 4 hours in the mold, the material is wire-cut into blocks and steam-cured under pressure in autoclaves for approximately 12 hours.

Emerging from the autoclave is a lightweight material with a sponge-like cellular structure, which is ready for shipping and use. AAC is approximately 1/5th the weight of ordinary concrete, with a dry bulk density ranging from 25 to 50 pcf (400 to 800 Kg/m³) and the specified compressive strength ranging from 300 to 1000 psi (2 to 7 MPa). The low density and cellular structure give AAC excellent thermal and sound insulation properties. It is also noncombustible and has low thermal conductivity. AAC can be easily cut, drilled and nailed by using normal hand tools.

RESEARCH SIGNIFICANCE

AAC is a well-recognized building material in Europe and around the world, with more than 300 production facilities currently operating worldwide but it is still new to the United States. The material properties and structural behavior of AAC have been

studied in Europe, and a large body of information is available; however, very little experimental work has been performed on American-produced material. Considerable technical developments are still needed in the U.S. in order to provide the user with a reasonable level of confidence. It is crucial to provide the basic engineering data to establish fundamental properties for design with AAC.

The need for a large-scale test program that incorporates multiple manufacturers is apparent. The test program would develop a database of material properties and structural behavior of American-made AAC, which would serve to establish AAC as a reliable engineered construction material. A comparison between AAC manufacturers would also be beneficial in providing designers with the essential information and tools necessary to enhance the design and production of AAC. Moreover, the information gained through the test program could be used to develop new ASTM test standards and building code design documents for AAC. This paper reports data based on a major study performed to evaluate the mechanical properties of AAC produced and marketed in the U.S. [2].

SCOPE OF STUDY

The plain AAC units were furnished by three manufacturers, namely Hebel, Ytong, and Contec. Production plants of the first two manufacturers were based in the U.S. whereas the third manufacturer produced the AAC in Mexico. Three different AAC grades (G1, G2, and G3) were produced by each of the manufacturers for testing. The mechanical properties of plain AAC that were investigated included the compressive strength, flexural strength, shear strength, and modulus of elasticity. Table 1 delineates the different material grades, nominal dry density limits, and the compressive strengths requirements according to ASTM C 1386-98 "Standard Specification for Precast Autoclaved Aerated Concrete (PAAC) Wall Construction Units" [3] for the AAC tested under this program. Test methods developed in this study are being proposed as new ASTM test standards for AAC.

EXPERIMENTAL PROGRAM

Compressive strength

Compressive strengths were determined per ASTM C 1386-98 [3]. Three samples, as shown in Figure 1, were cut using a table saw from the middle third of a standard AAC building unit (200 x 200 x 600 mm). Each cube was measured for length, width, and height.

Since the compressive strength is dependent on the direction of rise, the samples were tested both perpendicular and parallel to the direction of rise. The test setup is shown in Figure 2. Load was applied at a rate such that failure occurred within one to two minutes. Ultimate load and failure modes were recorded for each sample. The average compressive stress of the three cube specimens (100 x 100 x 100 mm) was calculated. The cubes were dried and the moisture content and dry bulk density were also

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calculated. The moisture content for the samples ranged from 8.1 to 11.3% by weight, which is within tolerances specified by ASTM C1386-98 of 5 to 15%.

Figure 3 shows the compressive strength by manufacturer, density, and direction relative to rise. The cubes tested perpendicular to the direction of rise consistently yielded higher compressive strength than the cubes tested parallel to the direction of rise for all grades tested with the exception of the Contec G1 and G2 samples. Figure 3 shows the ratios of the perpendicular to parallel test directions. Generally, the compressive strength increases as the density of the AAC increases. This was true for all grades test except for Ytong G3. It was found that Ytong G3 material consistently performed below the G2 material in all subsequent tests.

Regarding the ASTM recommended values, which are listed in Table 1, for the compressive strength of cubes tested perpendicular to rise, all manufacturers met the minimum required compressive strengths except Hebel G1 and Ytong G3 (Hebel G1 and Ytong G3 fell below the minimum acceptable values by 3.6 and 45%, respectively). Regarding the ASTM values for the average compressive strengths, all manufacturers failed to achieve 360 psi for G1. Also, Ytong and Contec failed to meet the average compressive strength for G3. The differences from the values published in the ASTM standard do not necessarily imply a deficiency in the material properties of the AAC tested, but shows that U.S. AAC may have somewhat different strength ranges for the densities produced. It should be pointed out that the values in Table 1, which are based on ASTM C1386-98, were developed based on data from European sources and are not representative of AAC made in the U.S. Hence, changes to the ASTM standard to reflect properties of AAC material manufactured in the U.S. are recommended.

Flexural tensile strength

The flexural tensile strength, or modulus of rupture (MOR), is an important property in the design of AAC walls and other structural elements that may be subjected to tensile stresses induced by various loading conditions. Two test methods were used to determine the MOR. For each method, six specimens were tested.

Method 1 allowed for three 50 x 50 x 200 mm prisms to be cut from each third of a standard building block (200 x 200 x 600 mm), having a shear span to depth (a/d) ratio of 1.25 ($a = 62.5$ mm, $d = 50$ mm). The prisms were then dried to 5 to 15% moisture by weight in a ventilated oven (70 °C). The standard building block size allowed testing in both the perpendicular and parallel directions, as shown in Figure 4. For the flexural tensile strength of Method 1, the setup, shown in Figure 5, consisted of a simple beam with three-point loading, a span of 125 mm, and a shear span of 62.5 mm. The loading was applied through steel rollers at a rate that caused failure in two to three minutes; however, strips to prevent bearing failures at the loading points were unnecessary with the small loads. The average flexural tensile strengths for Method 1, both perpendicular and parallel to rise, are shown in Figure 6. The MOR perpendicular to rise was, on average, approximately 30% of the compressive strength perpendicular to

rise. The MOR parallel to rise was, on average, approximately 36% of the compressive strength perpendicular to rise.

The MOR for Method 1 increased as the density of the material increased. This was true for all grades tested, except Ytong G3. For Ytong G3, this behavior was consistent with the compressive strength results. Interestingly, the MOR in the parallel direction was consistently higher than the MOR in the perpendicular direction. This behavior was not expected, as the material is typically stronger in the direction perpendicular to rise, as evident by the compressive strength results. However, this may be due to the direction of the planes of weakness in the AAC. When the specimens were loaded parallel to the direction of rise, the tensile forces developed perpendicular to the planes of weakness, pulling them apart. Conversely, when the tensile specimens were loaded perpendicular to the direction of rise, the tensile stresses developed parallel to the formation of the weakness planes, allowing the specimens to carry higher tensile forces before failure.

Method 2 typically allowed for two specimens (100 x 200 x 600 mm) to be cut from a standard block (200 x 200 x 600 mm), with a shear span to depth (a/d) ratio of 1.75 ($a = 175$ mm, $d = 100$ mm), as shown in Figure 7. These specimens can be loaded only perpendicular to the direction of rise, therefore inducing tensile stresses parallel to rise. The specimens were conditioned to 5 to 15% moisture by weight in a ventilated oven at 70 °C before testing. For Method 2, a modified version of the ASTM C 78 “Standard Test Method for Flexural Strength of Concrete” (4) setup was used to test the specimens. The test setup is shown in Figure 8. The flexural tensile strengths for Method 2 could only be determined for tensile stresses induced parallel to the direction of rise. The tensile stresses for Method 2 is shown in Figure 6 and were, on average, approximately 19% of the compressive strength perpendicular to rise.

For all manufacturers and grades, the MOR parallel to rise for Method 1 was consistently higher by 50 to 55% on average than the MOR parallel to rise for Method 2. The increase can be attributed to the size of the specimen, as well as the loading configuration, specifically point load for Method 1 versus two-point loading for Method 2.

Shear strength

A procedure to test shear strength was developed through trial and error, as well as based on previous research conducted at the University of Alabama at Birmingham. Analytical work was performed and various size specimens were tested before the decision was made to use a 200 x 300 x 600 mm AAC unit. It should be noted that this is not a standard-sized block; for this test, larger blocks (“mini-jumbos”) were needed to ensure shear failures. No saw cuts were required, only the conditioning of the specimens to the appropriate moisture content. Before testing, each block was dried to 5 to 15% moisture in a ventilated oven at 70°C.

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The shear strength of AAC was determined by the average of three full-size block specimens of dimensions 200 x 300 x 600 mm tested perpendicular to rise, as shown in Figure 9. The shear strengths of the three grades of AAC for Hebel, Ytong, and Contec are shown in Figure 10 for the specimens that were oven-dried to 5 to 15% moisture.

Because of the difficulty and time required to condition specimens, additional 200 x 300 x 600 mm specimens were tested for each grade of material at their natural moisture content. The specimens were not oven-conditioned and were tested at moisture contents ranging from approximately 15 to 25% by weight. The shear strengths of these specimens ranged from 55 to 132 psi (0.38 to 0.91 MPa). Interestingly, the shear strengths increased for the air-cured lab specimens for every grade of AAC (except Contec G1), even though they were tested at higher moisture contents. This is likely due to micro-cracking that may develop when the AAC blocks are subjected to extended periods of oven drying at 70 °C. However, because of the limited data, no solid conclusions could be drawn. Further testing is needed to determine if shear strength specimens should be tested at higher moisture contents.

It was found that the shear strength ranged from 8.9 to 25.3% of the compressive strength for the oven-cured specimens and 13.5 to 24.7% of the compressive strength for the lab-dried specimens. On average, the shear strength was approximately 17% of the compressive strength of the material.

Modulus of elasticity

The secant modulus of elasticity is the change in stress divided by the change in strain for two points. The two points used to calculate the modulus of elasticity were the $0.05f_{aac}$ and $0.33 f_{aac}$ stress levels, respectively. Three prisms (100 x 100 x 300 mm), cut as shown in Figures 10 and 11, were tested both perpendicular and parallel to rise for Hebel G1, Ytong G2, and Contec G2. The test setup is shown in Figure 13 and the loading cycle for each test is shown in Figure 14. Strains were recorded and the corresponding stresses and modulus of elasticity computed. A summary of the modulus of elasticity results is shown in Figure 15.

The stress-strain curves, plotted for the modulus of elasticity data, showed noticeable differences between the different AAC manufacturers. The Hebel G1 curves, both perpendicular and parallel to rise, were linear up to failure. The Ytong G2 curves, both perpendicular and parallel, were linear up to approximately 55% of the compressive strength before becoming nonlinear. The Contec G2 curves, like Hebel, proved to be more linear up to failure. However, some of the curves showed nonlinear behavior at approximately 60% of the compressive strength.

SUMMARY AND CONCLUSIONS

An experimental program was undertaken to evaluate the mechanical properties of plain AAC produced by different manufacturers. The mechanical properties for three

grades of AAC produced by Hebel, Ytong, and Contec were investigated. Specifically, the compressive strength, flexural tensile strength, shear strength, and modulus of elasticity were determined.

Information gained from this study is of benefit to the design engineer. Additionally, the test program provides useful information for the development of material, testing, and design standards for AAC. Test methods for flexural strength, shear strength, and modulus of elasticity were developed as part of this study and are being used for the development of new ASTM test standards for AAC. It is also recommended to revise the specified AAC strength values in ASTM C1386-98 to better reflect the material produced in the U.S.

The variation in the material properties with respect to the direction of rise indicates that further testing may be needed for both the perpendicular and parallel directions of AAC. Also, a study of the pore structure of the AAC material may aid in explaining the difference in the flexural tensile strengths between the perpendicular and parallel directions.

ACKNOWLEDGMENTS

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Table 1: ASTM C 1386 – 98 Physical Requirements

AAC Grade	Nominal dry Density ^a pcf (kg/m ³)	Density limits pcf (kg/m ³)	Average compressive strength psi (MPa)	Minimum compressive strength psi (MPa)
G1	25 (400)	22 (350)-28 (450)	360 (2.5)	290 (2.0)
G2	31 (500)	28 (450)-34 (550)	360 (2.5)	290 (2.0)
G3	37 (600)	34 (550)-41 (650)	725 (5.0)	580 (4.0)

a. The dry density is the average density of the material after being dried in a ventilated oven at 100 °C until no further weight loss occurs with successive drying.

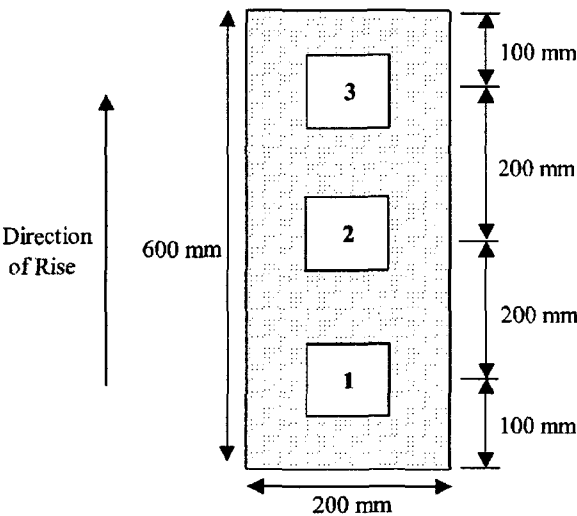


Figure 1: Cube location in a standard AAC building unit

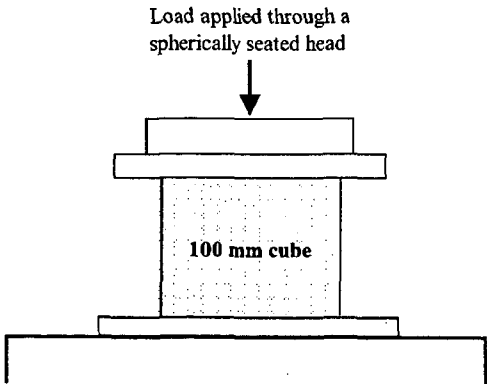


Figure 2: Test setup for compressive strength of AAC, 100 mm cubes

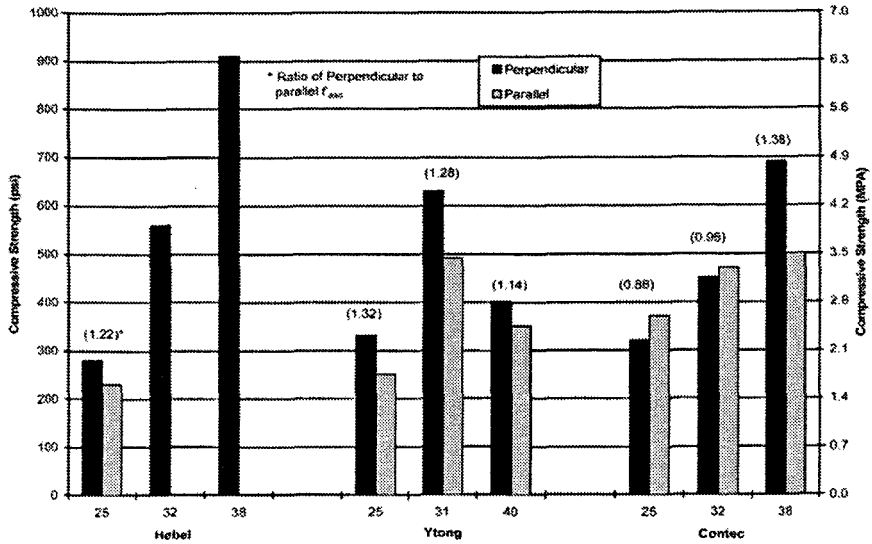


Figure 3: Compressive strength of AAC cubes versus density

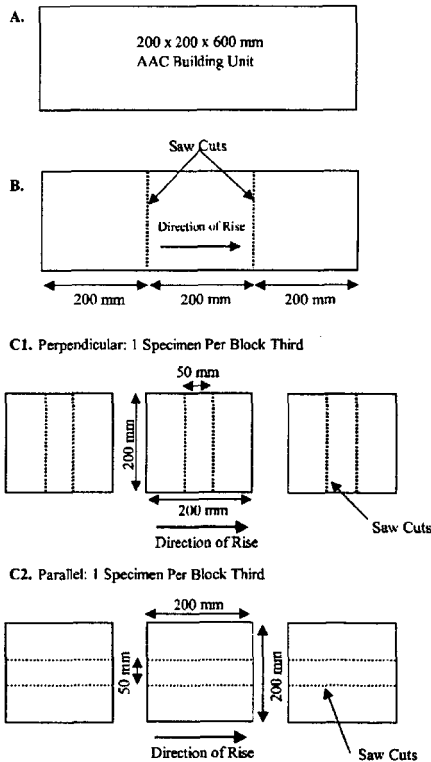


Figure 4: Flexural tensile strength specimens for Method 1

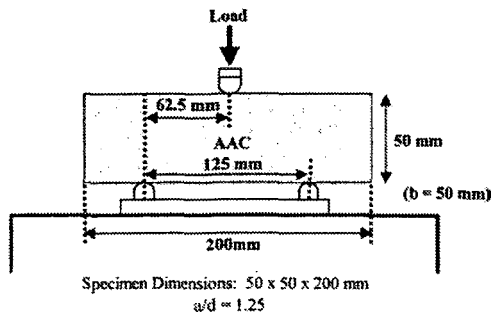


Figure 5: Test setup for flexural tensile strength for Method 1

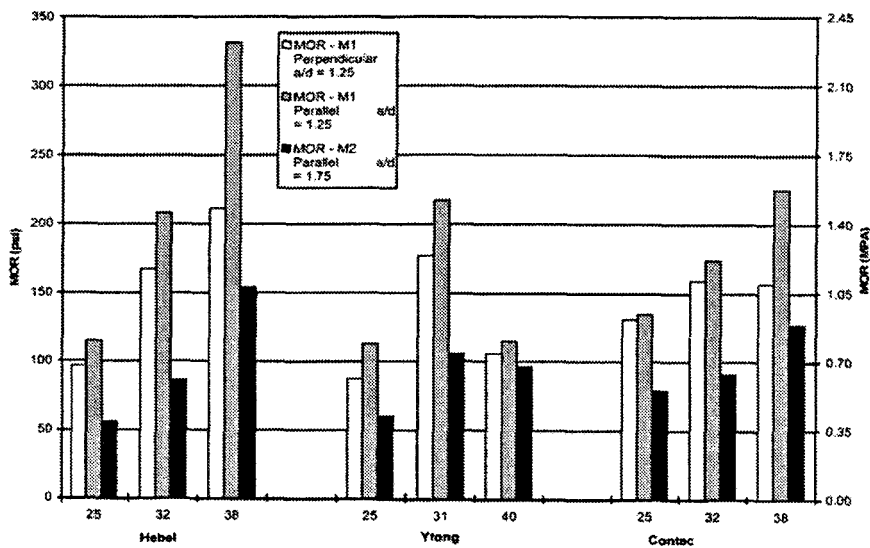
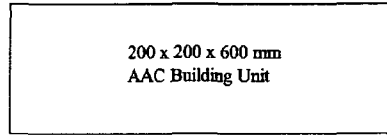
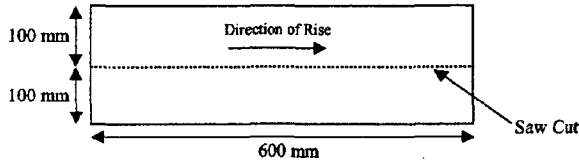


Figure 6: Test result comparison for Methods 1 and 2

A. Full-size Block



B. Saw Cut: 2 Specimens Per Block



C. Tensile Stresses Induced Parallel to the Direction of Rise

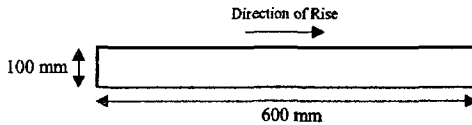


Figure 7: Flexural tensile strength specimens for Method 2

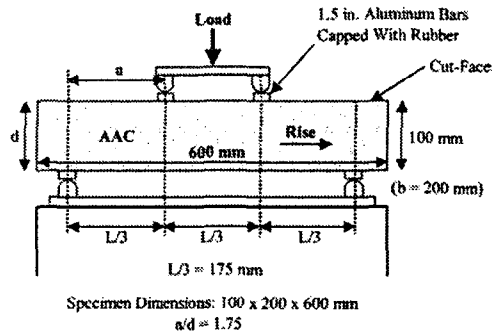


Figure 8: Test setup for flexural tensile strength for Method 2

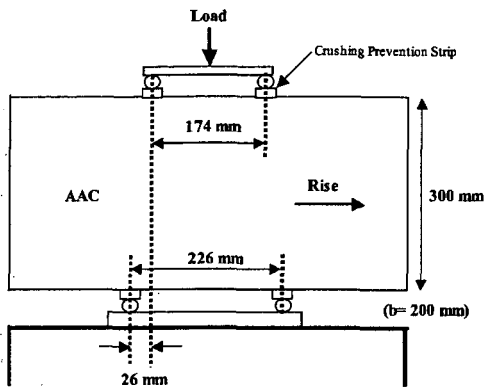


Figure 9: Test setup for shear strength test

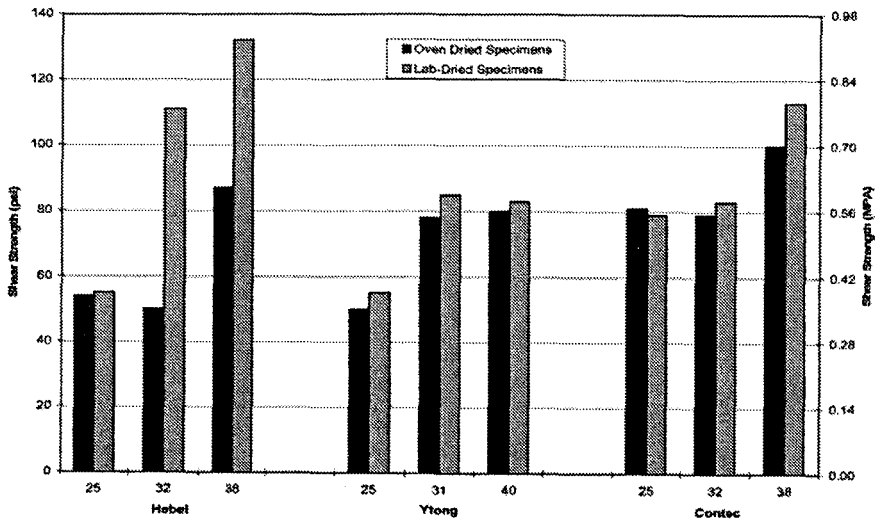
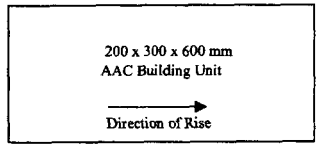
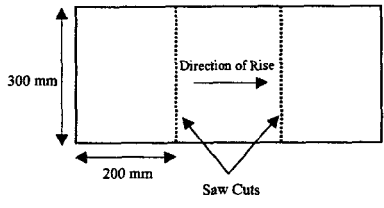


Figure 10: Direct shear test results

A. Mini-Jumbo Block



B. Saw Cuts



C. 1 Specimen Per Block Third

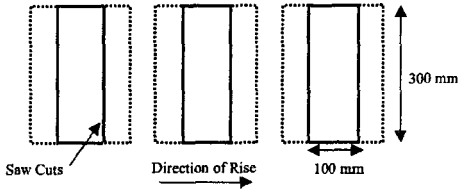
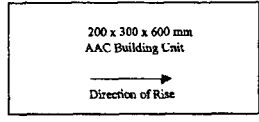
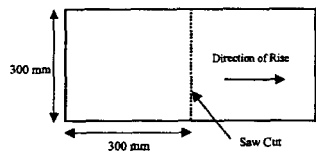


Figure 11: Modulus of elasticity test specimens, perpendicular to rise

A. Mini-Jumbo Block



B. Saw Cuts



C. 1 Specimen Per Block Half

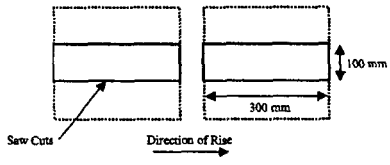


Figure 12: Modulus of elasticity test specimens, parallel to rise

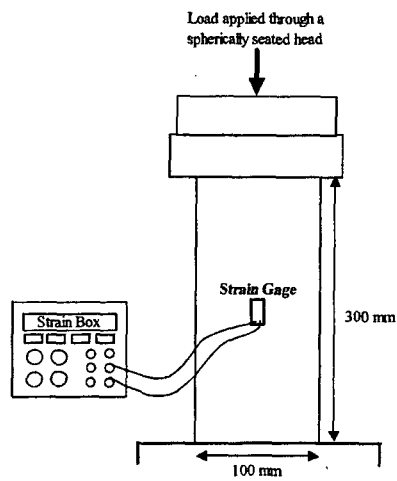


Figure 13: Test setup for modulus of elasticity

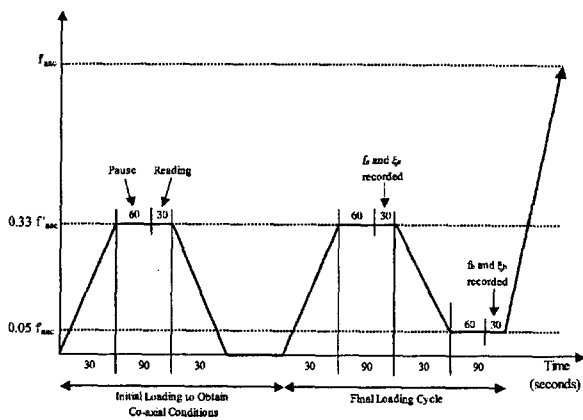


Figure 14: Loading cycle diagram for modulus of elasticity tests

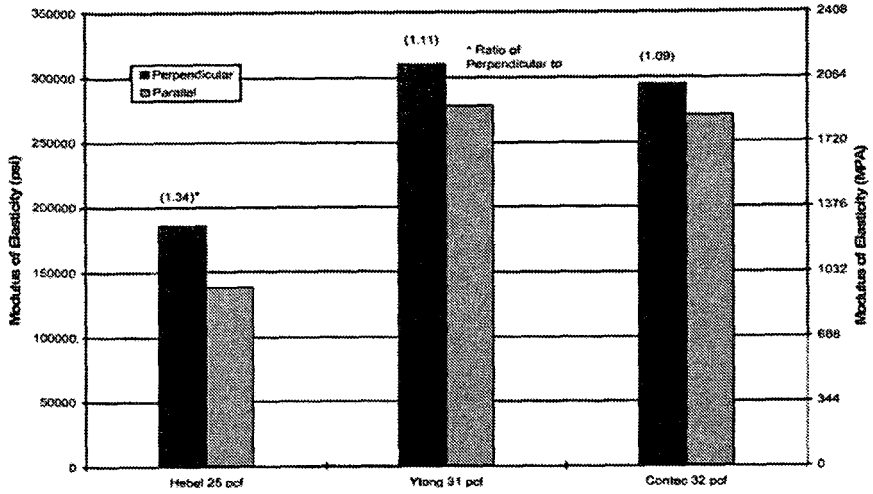


Figure 15: Test results for modulus of elasticity

Guide for Using Autoclaved Aerated Concrete Panels: I - Structural Design

by R. E. Barnett, J. E. Tanner, R. E. Klingner and F. H. Fouad

Synopsis: This paper is a summary of ACI 523.5R, which is a guide for using autoclaved aerated concrete panels. Its design provisions are non-mandatory, and are a synthesis of design recommendations from the Autoclaved Aerated Concrete Products Association, and from the results of research conducted at the University of Alabama at Birmingham (UAB), the University of Texas at Austin (UT Austin), and elsewhere.

This paper discusses the design equations associated with the various typical structural uses of autoclaved aerated concrete. Those uses include flexural, axial compression, shear, bearing, bond and development of reinforcement and special seismic design provisions.

The design provisions of this Guide are not intended for use with unreinforced, masonry-type AAC units. Design of those units is covered by provisions currently under development within the Masonry Standards Joint Committee.

Keywords: autoclaved aerated concrete; panels; precast; seismic; structural design

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INTRODUCTION

ACI 523.5R is a guide. Its design provisions are non-mandatory, and are a synthesis of design recommendations from the Autoclaved Aerated Concrete Products Association, and from the results of research conducted at the University of Alabama at Birmingham (UAB), the University of Texas at Austin (UT Austin), and elsewhere.

In chapter 3 of this Guide, the proposed design provisions are introduced in narrative form. In the appendices, more information is presented regarding specific design provisions (Appendix A), a commentary on those provisions, and a "super-commentary" with the technical justification for those provisions.

Loads for structural design of AAC elements should be taken from appropriate load codes, such as the local building code or ASCE 7. Understrength factors (Φ -factors) for AAC elements 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 justifiably identical to 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 concrete. They may even be higher, because the factory production of AAC leads to decreased variability in its mechanical characteristics compared to conventional concrete.

The equations throughout this document are given in both US Customary and SI units. The equations for US Customary units are designated as the Equation Number with an

“a” suffix. The equations for SI units are designated as the Equation Number with a “b” suffix. When the same equation is valid for both sets of units, only one equation is shown.

The design provisions of this Guide are not intended for use with unreinforced, masonry-type AAC units. Design of those units is covered by provisions currently under development within the Masonry Standards Joint Committee.

PROPOSED DESIGN PROVISIONS FOR REINFORCED AAC PANELS

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 reinforcement (with some modifications as noted below); stress-strain behavior of AAC and reinforcement; and equilibrium. The design strength of AAC in compression is based on a specified compressive strength, f'_{AAC} . Compliance with that specified compressive strength is verified by compressive strength testing, using ASTM C 1386. 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.

The modulus of elasticity for AAC is determined by

$$E_{AAC} = 6500 f'^{0.6}_{AAC} \quad (\text{Equation 1a})$$

$$E_{AAC} = 887.7 f'^{0.6}_{AAC} \quad (\text{Equation 1b})$$

The splitting tensile strength, f_{tAAC} is determined by

$$f_{tAAC} = 2.4 \sqrt{f'_{AAC}} \quad (\text{Equation 2a})$$

$$f_{tAAC} = 0.2 \sqrt{f'_{AAC}} \quad (\text{Equation 2b})$$

The modulus of rupture, f_{rAAC} , for AAC elements is taken as two times the splitting tensile strength, f_{tAAC} . However, if a section of AAC contains a horizontal leveling bed of conventional mortar, the value of f_{rAAC} is limited to 50 psi (345 kPa) at that section and if a section of AAC contains a bed joint using thin-bed mortar between AAC elements, the value of f_{rAAC} is limited to 80 psi (552 kPa) at that section.

Immediate deflections are calculated using an effective flexural stiffness (EI_e) corresponding to the unfactored moment (M_a). The effective flexural stiffness (EI_e) is 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.

Unless values are obtained by a more comprehensive analysis, the total long-term deflection (including the additional long-term deflection resulting from creep and shrinkage) of AAC flexural members is calculated the same as for immediate deflections, but with an effective modulus equal to $E_{AAC} / 1.5$.

Combinations of Flexure and Axial Load

AAC panels are designed for combinations of flexural and axial load using principles identical to those for conventional reinforcement. Nominal capacity is computed assuming plane sections; tensile reinforcement is assumed to be yielded; 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.

Because reinforced AAC panels usually have equal areas of tensile and compressive reinforcement, flexural capacity is usually “tension-controlled” or “under-reinforced.”

The factor β_1 is taken as 0.67 for AAC and the minimum reinforcement of flexural members is calculated by

$$As_{\min} = \frac{4 \sqrt{f'_{AAC}}}{f_y} bd \quad (\text{Equation 3a})$$

$$As_{\min} = \frac{0.33 \sqrt{f'_{AAC}}}{f_y} bd \quad (\text{Equation 3b})$$

Shear and Torsion

As with conventional reinforced concrete elements, the shear resistance of AAC elements 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.

$$\phi V_n \geq V_u \quad (\text{Equation 4})$$

$$V_n \geq V_{AAC} + V_s \quad (\text{Equation 5})$$

The shear resistance due to the AAC itself (V_{AAC}) is computed using the web-shear approach of ACI 318-02. The diagonal tension resistance of the AAC is expressed in terms of its specified compressive strength and principal tensile stresses, including the effects of axial loads, and is equated with this strength. This produces an expression for V_{AAC} in terms of the diagonal tension resistance of the AAC, and the axial load on the element.

For members subjected to shear and flexure only,

$$V_{AAC} = 0.8 \sqrt{f'_{AAC}} b_w d \quad (\text{Equation 6a})$$

$$V_{AAC} = 0.07 \sqrt{f'_{AAC}} b_w d \quad (\text{Equation 6b})$$

For members subject to axial compression,

$$V_{AAC} = 0.8 \sqrt{f'_{AAC}} \left(1 + \frac{N_u}{2000 A_g} \right) b_w d \quad (\text{Equation 7a})$$

$$V_{AAC} = 0.07 \sqrt{f'_{AAC}} \left(1 + \frac{N_u}{13.8 A_g} \right) b_w d \quad (\text{Equation 7b})$$

The shear resistance due to transverse reinforcement is computed based on the cross-sectional area of the transverse reinforcement crossing a hypothetical 45-degree crack in the AAC. It may also be limited by bond and development of the reinforcement.

The capacity of the shear reinforcement perpendicular to the axis of the member is calculated by

$$V_s = \frac{A_v f_y d}{s} \quad (\text{Equation 8})$$

but is limited to the bearing capacity of the AAC on the longitudinal reinforcement, which is given by

$$V_{sh} = d_{long} s f'_{AAC} \quad (\text{Equation 9})$$

Torsion resistance capacities for AAC have not been determined, therefore AAC is not currently used in torsion resistant applications.

In non-seismic loading combinations, for shear friction reinforcement placed perpendicular to the shear plane, the shear strength is calculated by,

$$V_n = A_v f_y \mu \quad (\text{Equation 10})$$

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In seismic loading combinations, the shear strength of AAC is calculated by either,

$$V_n = N_u \mu, \quad (\text{Equation 11})$$

where N_u is the factored normal force at the considered shear plane, or the direct shear strength of the joint. μ is 1.0 when AAC is placed against conventional Type "S" or Type "N" leveling mortar and 0.75 when placed against AAC.

The shear strength, V_{AAC} , for in-plane loading on walls is taken as the lesser of

$$V_{AAC} = 0.86 \ell_w h \sqrt{f'_{AAC}} \sqrt{1 + \frac{N_u}{2.4 \sqrt{f'_{AAC}} \ell_w h}} \quad (\text{Equation 12a})$$

$$V_{AAC} = 0.07 \ell_w h \sqrt{f'_{AAC}} \sqrt{1 + \frac{N_u}{0.2 \sqrt{f'_{AAC}} \ell_w h}} \quad (\text{Equation 12b})$$

or

$$V_{AAC} = \frac{0.17h f'_{AAC} \ell_w^2 h_w}{h_w^2 + (\frac{3}{4} \ell_w)^2} \quad (\text{Equation 13a})$$

$$V_{AAC} = \frac{170000h f'_{AAC} \ell_w^2 h_w}{h_w^2 + (\frac{3}{4} \ell_w)^2} \quad (\text{Equation 13b})$$

For shear walls with AAC panels oriented vertically, the nominal in-plane shear strength and flexural strength shall be determined assuming that vertical cracks exist at each vertical joint. The shear capacity shall be determined using Equation 7 if the panel height divided by the panel width exceeds 3. It shall be permitted to design assuming vertical cracks at every third joint, using Equation 13.

Where the factored shear force V_u exceeds the shear strength ϕV_{AAC} , the horizontal shear reinforcement shall be provided to satisfy Equation 4 and Equation 5, where the shear strength V_s is computed by

$$V_s = \frac{A_v f_y d}{s_2}, \quad (\text{Equation 14})$$

where A_v is the area of deformed horizontal reinforcement embedded in grout within a distance s_2 and distance d is in accordance with ACI 318, Section 11.10.4.

Bond and Development of Reinforcement

Reinforcement in AAC panels consists of welded-wire cages installed when the panels are produced, and deformed reinforcement installed in 3- to 4- in. grouted cores as the panels are erected. The maximum ratio of vertical reinforcement to area of a grouted cell shall be 3%.

Splices of longitudinal reinforcement are not permitted in potential plastic hinge zones.

Bond and development requirements for deformed reinforcement in grout are identical to those used for concrete construction. Given the small sizes of deformed bars used in AAC construction, bond between the grout and the AAC itself does not govern the bond capacity.

Bond and development requirements for welded-wire cages embedded in AAC are quite different from those for conventional concrete, however. Because the welded-wire cage has a corrosion-resistant coating and the wires are not deformed, 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 spacings, 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. 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 welded-wire cage (Figure 1).

This maximum force in the welded-wire cage can limit the flexural capacity of a reinforced AAC panel.

Factory-installed reinforcement embedded in AAC shall be designed to satisfy either Equation 15 or 16. The spacing of cross-wires, s_{cross} , in factory installed reinforcement embedded in AAC is limited to less than or equal to the value required to satisfy Equation 15.

$$s_{max} = \frac{0.85 d \cdot d_{cross} \cdot l_{cross} \cdot f'_{AAC}}{V_u / \phi} \quad (\text{Equation 15})$$

The number of cross-wires within a distance of one sixth of the clear span of the panel, measured in each direction from each support, shall equal or exceed the value required to satisfy Equation 16. In that equation, a is the shear span or one sixth of the clear span of the panel. In other sections, the spacing shall not exceed $2s_{min}$.

$$n_{cross,min} = \frac{V_u / \phi}{0.85 d \cdot d_{cross} \cdot l_{cross} \cdot f'_{AAC}} a \quad (\text{Equation 16})$$

Precast Concrete Structural Integrity

Where precast AAC elements form floor or roof diaphragms, the following provisions shall apply:

1. The nominal shear strength at the interface of dissimilar materials shall be based on adhesion at diaphragm joints and shall be computed as the product of the contact area of grout and AAC and the shear strength of a grout and AAC joint plus the product of the contact area of thin-bed mortar and AAC and the shear strength of thin-bed mortar. The shear strengths of joints between thin-bed mortar and AAC and grout and AAC are 18 psi (0.13 MPa) and 36 psi (0.25 MPa), respectively.
2. The nominal shear strength of AAC floor and roof diaphragms shall be based on a truss model subject to the following minimum provisions:
 - a. Compression struts shall not be permitted to cross panel joints and shall intersect with tension ties in grouted keys and tension ties (chords) in ring/bond beams.
 - b. Tension ties shall consist of the reinforcement in grouted keys or in a ring/bond beam. The reinforcement in the grouted keys shall be hooked around the longitudinal reinforcement in the ring beam with standard 90-degree hooks oriented in a vertical plane.
 - c. Compression struts shall be defined within the panel. Their dimension perpendicular to the plane of the panel shall be taken equal to the thickness of the panel. Their dimension in the plane of the panel, measured perpendicular to the axis of the strut, shall be taken as 6 in. The nominal strength of compression struts shall be calculated as the product of 85 percent of the specified compressive strength of the AAC and the cross-sectional area of the strut.
 - d. The nominal strength of the tension ties shall not exceed the product of the cross-sectional area of the reinforcement and the specified yield strength of the reinforcement.
3. The nominal shear strength shall be based on dowel action of reinforcement in the grouted keys perpendicular to the lateral load. The nominal shear strength shall be computed as the product of 60 percent of the cross-sectional area of the reinforcement and the specified yield strength of the reinforcement.

Vertical tension tie requirements of ACI 318 Section 7.13.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with the following:

- a. Precast columns shall not be made of AAC;

- b. Precast wall panels that comprise shear walls shall be connected at wall intersections and at locations of longitudinal reinforcement;
- c. When design forces result in no tension at the base, the ties required by (b) above, shall be permitted to be anchored into an appropriately reinforced concrete slab on grade.

Except for sliding shear resistance in a shear wall, connection details that rely solely on friction caused by gravity loads shall not be used.

For precast autoclaved aerated concrete bearing wall structures three or more stories in height, the following minimum provisions shall apply.

- a. Longitudinal and transverse ties shall be provided in floor and roof systems, and shall be designed to transfer shear to lateral force-resisting elements. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 2 ft of the plane of the floor or roof system. Longitudinal ties shall only be required parallel to the direction of span of the panels.
- b. Longitudinal ties parallel to floor or roof slab spans shall be spaced not more than 10 feet on centers. Provision shall be made to transfer forces around openings.
- c. Transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.
- d. Ties around the perimeter of each floor and roof shall resist the design loads acting at that level.
- e. Continuous vertical reinforcement in AAC shear walls shall be sufficient to resist the design moments.

Bearing

To prevent local crushing of the AAC, nominal stresses are limited to f'_{AAC} . 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.

The design bearing strength is limited to $\phi(0.85 f'_{AAC} A_1)$, similarly as given in ACI 318 for conventional concrete.

Unless shown by test or analysis that performance will not be impaired, the following minimum requirements shall be met:

- a. Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least 1/180 of the clear span, but not less than:

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For solid or hollow-core slabs	2 in.
For beams or stemmed members	3 in.
For autoclaved aerated concrete panels	2 in.

- b. Bearing pads at unarmored edges shall be set back a minimum of 1/2 in. from the face of the support, or at least the chamfer dimension at chamfered edges.

The requirements of ACI 318 Section 12.11.1 do not apply to the positive bending moment reinforcement for statically determinate precast AAC members. At least one-third of such reinforcement, however, shall extend to the center of the bearing length.

Special Provisions for Seismic Design

The provisions of this section apply to design of intermediate AAC structural walls and their associated horizontal diaphragms to resist forces induced by earthquake motions.

1. The design shear force V_e shall be determined from consideration of the maximum forces that can occur in an AAC element. Forces in the longitudinal reinforcement shall be determined assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.
2. The horizontal diaphragm shall be designed to resist the design shear force, V_e . Design according to ACI 318, Section 16.5.1.2.2 is not permitted.

The lateral load between horizontal diaphragms and AAC structural walls shall be transferred through connectors embedded in grout in accordance with ACI 318, Section 16.

Summary and Conclusions

The “Proposed Design Provisions for Reinforced AAC Panels” is formatted in such a way as to facilitate its use in conjunction with ACI 318. Many current provisions apply directly to AAC, however this guide gives specific direction to the designer for the design of reinforced AAC based on its behavior.

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Notations

A_g	=	gross area of section, in ² (mm ²)
A_s	=	area of steel, in ² (mm ²)
A_v	=	area of shear reinforcement within a distance s , in ² (mm ²)
b	=	width of compression face of member, in (mm)
d	=	distance from extreme compression fiber to centroid of tension reinforcement, in (mm)
d_{cross}	=	diameter of cross-wires, in. (mm)
d_{long}	=	diameter of longitudinal reinforcement, in. (mm)
E_{AAC}	=	modulus of elasticity of AAC, psi (MPa)
f_{AAC}	=	specified compressive strength of AAC, psi (MPa)
f'_g	=	specified compressive strength of grout, psi (MPa)
f_y	=	specified yield strength of nonprestressed reinforcement, psi (MPa)
f_{rAAC}	=	modulus of rupture of AAC, psi (MPa)
f_{tAAC}	=	splitting tensile strength of AAC by ASTM C 1386, psi (MPa)
h	=	overall thickness of member, in (mm)
h_w	=	total height of wall from base to top, in (mm)
l_{cross}	=	length of reinforcement bearing on AAC, in. (mm)
ℓ_w	=	horizontal length of wall, in (mm)
N_u	=	factored axial load normal to cross-section occurring simultaneously with V_u , lb (N)
s	=	spacing of shear reinforcement in direction parallel to longitudinal reinforcement, in (mm)
V_{AAC}	=	nominal shear strength provided by AAC, lb (N)
V'_{sb}	=	maximum usable shear strength provided by each wire of shear reinforcement, lb (N)
V_n	=	nominal shear strength, lb (N)
V_s	=	nominal shear strength provided by shear reinforcement, lb (N)
V_{sb}	=	bearing capacity of the AAC on the anchorage wires, lb (N)
V_u	=	factored shear force at section, lb (N)
ρ_{1386}	=	air-dried density of AAC by ASTM C 1386, lb/ft ³ (kg/m ³)
w_{strut}	=	horizontal projection of the width of the compression strut, in. (m)

References

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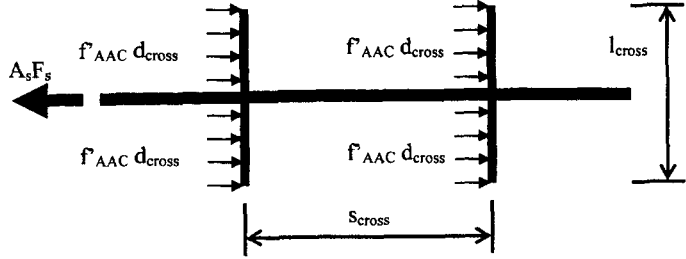


Figure 1: Bond mechanism of welded-wire cage in AAC

Structural Testing for Validating Reinforced AAC Design Provisions in the U.S.

by F. H. Fouad and J. Dembowski

Synopsis: Autoclaved aerated concrete (AAC) is a lightweight uniform cellular material, first developed in Sweden in 1929. Since that time, plain and reinforced AAC building components have been widely used in Europe and other parts of the world. Until recently, however, AAC was relatively unknown to the United States precast construction market. Today, AAC prefabricated elements are gaining rapid acceptance in the United States due primarily to increasing energy cost, environmental concerns, and the ease of construction using AAC elements. Although AAC is a well-recognized building material in Europe, very little research work has been done on U.S.-produced AAC products.

The primary objective of this work was to study the structural behavior of U.S.-made reinforced AAC elements. The laboratory test program included most commonly used reinforced AAC elements: floor panels, lintels, and wall panels. Two U.S. manufacturers supplied the AAC elements. Floor panels and lintels were tested in bending, whereas the wall panels were tested under axial or eccentric loading. The ultimate load capacity, cracking, deflection, and failure mode were observed and recorded for each test. The results provide a database that will be used to refine the analytical methods for the structural design of reinforced AAC elements. This information is needed to enhance AAC design methodologies and lay the foundation for establishing AAC as a reliable engineered construction material in the U.S.

Keywords: autoclaved aerated concrete; floor panel; lintel; reinforced; wall panel

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INTRODUCTION

Autoclaved aerated concrete (AAC) is a lightweight uniform cellular material, first developed in Sweden in 1929, and based on a patented process by Johan Eriksson. AAC is a well-recognized building material in Europe and around the world, with more than 300 production facilities currently operating worldwide but it is still new to the United States. The first major production plant was not constructed in the U.S. until 1996. Today, AAC is gaining rapid acceptance as a new building product in the U.S. as a result of the increasing importance placed on energy, since energy savings are realized both in the production process of AAC and in the thermal insulation properties of the AAC finished product [1]. The rising cost of lumber and increasing environmental concerns have also played a role in the interest surrounding AAC.

AAC production is very sensitive to the quality of the materials used in the concrete mix and their proportions. The raw materials consist of Portland cement, finely ground sand, and lime. These materials are mixed with water and a small amount of aluminum powder and cast into steel molds where the steel reinforcing cages are secured. Hydrogen gas, a product of the reaction between the cement hydration products and the aluminum powder, causes the material to rise in the mold, creating macroscopic air cells throughout the material. After approximately 3 to 4 hours, the material is removed from the molds and wire-cut into the required sizes and shapes. After cutting, the AAC product is steam-cured under pressure in autoclaves for approximately 8 to 12 hours.

After autoclaving, the material is ready for shipping and use. AAC has a lightweight and sponge-like cellular structure which is approximately $1/5^{\text{th}}$ to $1/3^{\text{rd}}$ the weight of ordinary concrete, with a dry bulk density ranging from 25 to 50 pcf (400 to 800 Kg/m³) and the specified compressive strength ranging from 300 to 1000 psi (2 to 7 MPa). The low density and cellular structure give AAC excellent thermal and sound insulation properties. It is also noncombustible and has low thermal conductivity. Unreinforced AAC can be easily cut, drilled and nailed by using normal hand tools.

The material properties and structural behavior of AAC have been studied in Europe, and a large body of information is available; however, very little experimental

work has been performed on American-produced material. It is crucial to provide the basic engineering data to establish fundamental properties for design with AAC.

RESEARCH SIGNIFICANCE AND SCOPE

The primary objective of this work was to study the structural behavior of American-made reinforced AAC elements. The laboratory test program included most commonly used reinforced AAC elements: floor panels, lintels, and wall panels. Two U.S. manufacturers, namely Hebel and Ytong, supplied the AAC elements. Floor panels and lintels were tested in bending, whereas the wall panels were tested under axial loading. The ultimate load capacity, cracking, deflection, and failure mode were observed and recorded for each test. Tests were performed on AAC material of Grade G2, which is most commonly used for these elements. Table 1 delineates the different material grades, nominal dry density limits, and the compressive strengths according to ASTM C 1386-98 "Standard Specification for Precast Autoclaved Aerated Concrete (PAAC) Wall Construction Units" [2].

Plain reinforcing steel wire conforming to ASTM A 82 is used to construct the reinforcing cage or mat by spot welding. It should be noted that the mechanism of force transfer in reinforced AAC is quite different than in conventional reinforced concrete due to the lack of bond between the steel and AAC. Further the steel reinforcement has a corrosion-resistant coating which further degrades the bond between steel and concrete. As result, force transfer is attained through the mechanism of bearing of cross wires against the AAC. This fundamental difference should be considered in assessing the structural behavior of reinforced AAC elements.

Information from this study provides a better understanding of the structural behavior of reinforced AAC elements, and the test data obtained can be used to verify or refine proposed analytical methods for the design of reinforced AAC. Improved design methodologies for the material are needed to establish AAC as a reliable engineered construction material in the U.S.

EXPERIMENTAL PROGRAM

Reinforced AAC Floor Panels

Twelve reinforced floor panels of AAC Grade G2 supplied by Ytong were tested in flexure under transverse loading. The longitudinal steel reinforcement, floor thickness, and spans were variables in the study. A nominal compressive strength of 720 psi (5.0 MPa) was reported for the G2 material of the wall panels. Two different floor panel thicknesses were tested. The 6-inch (150 mm) thick floor panels were tested at a span of 12.5 feet (3810 mm) and the 8-inch (200 mm) thick floor panels were tested at a span of 16.5 feet (5030 mm). For both the 6 and 8-inch thick panels, three different reinforcing schemes were used with varying amounts of longitudinal steel and a standard cross-wire design. A summary of the floor panel test program is shown in Table 2. Figure 1

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provides the reinforcement pattern for the 6-inch (150 mm) floor panels, which is identical to the reinforcement pattern of the 8-inch (200 mm) panels.

The test setup for the AAC floor panels consisted of four-point loading at the quarter points in accordance with ASTM C 1452-00 "Standard Specification for Reinforced Autoclaved Aerated Concrete Elements" [3]. A detailed diagram of the floor panel setup is shown in Figure 2. Deflection measurements were taken at the mid-span and quarter points of the panels.

Loading was applied in approximately 500-pound (2.22 kN) increments and halted as cracks formed, just long enough to detect and document the crack locations and widths. Selected panels were unloaded after the first and second cracks formed in order to record any permanent deflection that may have occurred; otherwise, the panels were continuously loaded after each 500-pound (2.22 kN) load increment. The loading was also halted at 1500, 3000, and 4500 pounds (6.67, 13.34, and 20.02 kN) for 5 minutes to check the overall setup and to measure crack widths. When failure appeared imminent, the potentiometers and dial gages were removed. The load was then applied continuously at a rate of about 500 pounds (2.22 kN) per 30 seconds until failure.

A summary of the test results is provided in Tables 3 and 4. Once the panel cracked, the stress carried by the AAC is transferred to the reinforcing steel. The cracking load as a percentage of the maximum load ranged from 26 to 43% and is, on average, approximately 30% of the failure load. The variation in the cracking load was a function of the longitudinal steel reinforcement, the flexural tensile strength of the material, and the thickness of the floor panel and corresponding test span. Generally, for the same AAC grade and panel thickness, the cracking loads increased as the number of longitudinal bars increased and were highest for the floor panels with the most longitudinal steel reinforcement. The increase in cracking load is most likely due to a prestressing effect created by the longitudinal reinforcement as a result of the autoclaving process.

All of the AAC floor panels demonstrated similar cracking patterns and failure modes. The initial cracks were flexural cracks that occurred within the mid-span of the panels. As the loading increased, the cracks grew, and new cracks developed. Some of the flexural cracks began to "T" off at varying loads between the different floor panels tested. The depth of the horizontal portion of the "T" cracks was consistently 2 to 4 inches from the top, or compression face, of the panel, depending on the thickness of the floor panel. The horizontal portion of the "T" crack sometimes extended 3 to 5 inches on both sides of the flexural crack. Cracks, which began as flexural cracks, appeared in the shear span at approximately 30 to 50% of the maximum load for the 8-inch thick floor panels, while the 6-inch (150 mm) thick floor panels had no cracks in the shear region. After running approximately 4 to 5 inches (100 to 125 mm) vertically, the cracks curved towards the middle of the span under the influence of shear. The cracks then ran another 5 to 10 inches (125 to 250 mm) inclined.

As shown in Table 3, the average ultimate capacities of the 6-inch Ytong panels tested at 12.5-foot (3810 mm) spans ranged from 3920 pounds (17.44 kN) for samples with four 7 mm longitudinal tension bars to 6010 pounds (26.73 kN) for samples with ten 7 mm longitudinal tension bars. The average ultimate capacities of the 8-inch Ytong panels tested at 16.5-foot (3810 mm) spans ranged from 4238 pounds (18.85 kN) for samples with four 7 mm longitudinal tension bars to 8026 pounds (35.70 kN) for samples with ten 7 mm longitudinal tension bars. Load versus deflection data, representing the flexural stiffness of the floor panel, were plotted but are not included herein due to space limitations [4]. The ratio of the ultimate load at failure, $P_{\max}$ (or $w_{\max}$), to the load at first crack, P_{crack} (or w_{crack}), referred to herein as the safety factor, is given in Table 3 with values ranging from 3.1 to 4.0 for the panels tested.

The uniformly distributed load at first crack, w_{crack} , exceeded the superimposed design load specified by the floor panel manufacturer in all cases, except for the 8-inch (200 mm) thick panels containing 4 longitudinal steel bars (Table 3). In this case w_{crack} fell below the specified value by approximately 11 %, which may be attributed to the relatively lower steel reinforcement ratio for this size panel. Panel deflections at first crack, as shown in Table 4, were below the arbitrarily specified limit of Span/300, except for the 6-inch (150 mm) thick panels with 10 longitudinal bars on 12.5-foot (3810 mm) spans where the deflections exceeded the specified limit by about 9%. The high reinforcement ratio and short span resulted in a cracking load much greater (about 33%) than the superimposed design load specified by the manufacturer. However, under the actual superimposed design load as specified by the manufacturer, the deflections were well below the specified limit.

The floor panels exhibited large vertical deflections of 2 to 3 inches (50 to 150 mm) and considerable crack widths of 2 to 4 mm near failure. Yet, all floor panels exhibited a nonviolent ductile primary failure with crushing on the compression face. Three of the four floor panels with ten 7 mm longitudinal steel reinforcing bars exhibited a sudden, violent secondary pullout failure. It should also be noted that all floor panels failed at a cross bar or a stand used to hold the reinforcement in place. This is likely due to the stress concentrations at these points.

In addition to the twelve panels tested, one of the 6-inch (150 mm) thick floor panels was turned upside down, such that the tensile steel was located at the top, and tested. In this configuration, the panel had four 7 mm longitudinal steel bars in the bottom mat and six 7 mm longitudinal bars in the top mat. The purpose of testing the panel in this configuration was to determine the effectiveness of the compression steel on the ultimate flexural capacity. The additional two bars in the compression mat did not improve the ultimate flexural strength of the panel. It was observed that the panel reached the same strength as other panels with the same type of bottom reinforcement of four 7 mm longitudinal bars.

Reinforced AAC Lintels

A total of 24 lintels supplied by Hebel and Ytong were tested in flexure under transverse loading. The test lintels were of Grade G2 material and varied in cross sectional size, span, and reinforcement scheme as shown in Table 5. Hebel lintels had a U-shaped stirrup reinforcing pattern, while Ytong used double reinforcing mats similar to the floor panel reinforcement, but placed vertically in opposite sides of the cross section. Figure 3 shows a typical reinforcement layout for Hebel lintels.

A diagram of the lintel test setup is shown in Figure 4. Four-point loading at the quarter points was used in accordance with ASTM C 1452-00 [3]. Deflection measurements were taken at mid-span. The load was applied in 500-pound increments and halted as cracks formed, just long enough to detect and document the crack locations and widths. Selected lintels were unloaded after the first and second cracks formed in order to determine if a permanent deflection has occurred; otherwise, the lintels were continuously loaded after each 500-pound load increment. When failure appeared imminent, the potentiometer and dial gages were removed, and the load was continuously applied at a rate of about 500 pounds per 30 seconds until failure.

A summary of the lintel test results is provided in Tables 6 and 7 for the Hebel and Ytong lintels, respectively. Once the lintels cracked, the stresses carried by the AAC were transferred to the reinforcing steel. For all lintels tested, the load at first crack was approximately 15 to 42% of the maximum load carried. The average cracking load was approximately 36% of the failure load for Hebel lintels, and approximately 26% of the failure load for Ytong lintels.

The flexural stress at first crack was calculated for Hebel and Ytong lintels by using the observed cracking load. For Hebel lintels, the stress at first crack ranged from 285 to 388 psi (1.97 to 2.68 MPa) (Table 6). For Ytong lintels, the stress at first crack ranged from 96 to 178 psi (0.66 to 1.23 MPa) (Table 7). The stresses at first crack were higher for Hebel lintels because the actual compressive strength of the Hebel AAC was higher than Ytong's. Control cube specimens produced compressive strengths of 900 and 580 psi for the Hebel and Ytong AAC materials, respectively. Nevertheless, no apparent trends could be established from the stress at first crack data for the lintels of either manufacturer.

The majority of the lintels exhibited approximately symmetrical cracks about the centerline, and no difference in cracking was observed between the Hebel and Ytong lintels. The first crack was usually a flexural crack that occurred within the mid-span of the lintel. The ensuing cracks occurred either in the mid-span or shear span of the lintel, depending on the cross-sectional area and the span. The cracks in the shear span would curve inward toward the mid-span. As the loading increased, the cracks grew, and new cracks developed. Some of the cracks began to branch at loads ranging from 4000 to 7000 pounds (17.79 to 31.14 kN), depending on the cross-section. For deeper lintels, this range was even higher. This "branching" was common in the Ytong lintels, but occurred in only one of the Hebel lintels. It is important to note that these cracks were not like the

“T” cracks associated with the floor panels, which would split and run horizontally. In the lintels, the cracks would branch and continue running vertically at a slight angle.

The Hebel and Ytong lintel deflections at first crack ranged from 0.04 to 0.15 inches (1 to 4 mm), and were well below the acceptable limit of span divided by 300. Hebel lintel deflections at first crack were approximately one half of the allowable limit, on average, and Ytong lintel deflections at first crack were approximately one third of the allowable limit, on average.

The ultimate loads for Hebel lintels ranged from 8180 to 23,030 pounds for the different spans and cross-sections tested. For Ytong, the ultimate loads ranged from 7730 to 27,002 pounds (34.38 to 75.63 kN) for the different spans and cross sections tested. For the deeper Ytong lintels, the amount of longitudinal tension steel also varied and affected the ultimate capacities of the lintels. The average ultimate capacities of the Hebel lintels were higher than the Ytong lintels for the same cross section size and test span. Hebel lintels were approximately 36% higher for the 8 by 8-inch lintels tested at a 3-foot span (914 mm), and approximately 24% higher for the 8 by 8-inch (200 by 200 mm) lintels tested at a 6-foot (1830 mm) span. The difference between the ultimate capacities of the Hebel and Ytong lintels is due to the disparity in the strength of the AAC material, as evidenced by Hebel’s control block compressive strength of 900 psi compared with Ytong’s control block compressive strength of 580 psi.

Although the lintels exhibited vertical deflections and crack widths as large as 1.5 and 0.08 inches (38 and 2 mm), respectively, near failure, as with the floor panels, all failures were nonviolent and ductile. Every lintel, except the 2 YL 8-24-10, failed in the mid-span from the crushing of the AAC on the compression face. Visual observations of the failures were made, and no apparent signs of the compression steel buckling were detected. The Ytong 2 YL 8-24-10 developed so many cracks in the shear span that it failed mildly in shear under gradual increase in load.

One of the Hebel lintels, 2 HL 8-8-5 Down (Table 6), was tested upside down, with the 4 - 8 mm longitudinal steel bars placed in the top compression face and the 2 - 8 mm longitudinal bars placed in the bottom tension face. The test results showed that the cracking load was reduced by 38%, and the ultimate capacity was reduced by 30% when compared with the same lintels tested with the same steel reinforcement in the opposite faces. This demonstrated the effectiveness of having two additional longitudinal bars in the tension face. On the other hand, the additional two bars in the compression zone had little effect on the ultimate capacity of the lintel. Interestingly, the deflections at first crack were approximately the same for this size lintel tested with either reinforcement schemes.

Reinforced AAC Wall Panels

A total of six Ytong vertical wall panels of AAC Grade G2 were tested under axial or eccentric compression. A description of the test specimens is given in Table 8. Three different wall sizes were tested: 6, 8, and 9.5-inch (150, 200, and 240 mm) thick.

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The 6-inch (150 mm) thick panels were loaded concentrically, whereas eccentric loading was used for the 8 and 9.5-inch (200 and 240 mm) thick panels. A diagram of the reinforcement for the 6-inch thick wall panels is shown in Figure 5. The 8 and 9.5-inch thick wall panels had the same reinforcement scheme as shown in Figure 6.

Details of the eccentrically loaded test setup are shown in Figure 7. For the 8 and 9.5-inch thick wall panels, the load was applied with an eccentricity of $t/6$ per ASTM E 72-98 "Standard Test Method for Conducting Strength Tests of Panels for Building Construction," [5] where t is the thickness of the wall panel. The panel face closer to the applied load was referred to as the inside face, and the opposite face was referred to as the outside face. As load was applied, the vertical and horizontal deflection was measured. Load was applied in 20,000-pound (89 kN) increments up to 60,000 pounds (267 kN), then in 10,000-pound (45 kN) increments until failure. The failure modes of the wall panels were observed and recorded. Test results are provided in Table 9.

The average compressive strength of the 6-inch (150 mm) panels was approximately 456 psi (3.14 MPa), which compared favorably with the average compressive strength of the wall panel control block of 480 psi (3.31 MPa). Additionally, the nominal capacity, P_n , of the 6-inch (150 mm) wall panel under concentric loading was calculated, considering the length effect, to be 69,118 pounds (307 kN), which compared favorably (only 5% difference) to the average capacity from the panel tests, $P_{max} = 65,720$ pounds (292 kN). The small difference can be attributed to some slight eccentricity during testing. More importantly, however, this seems to indicate that the longitudinal steel reinforcement has very little effect on the panel's axial load carrying capacity.

All of the wall panels exhibited a mild, nonviolent ductile failure. The panels failed in compression when the concrete shell covering the reinforcement mat cracked. For the eccentrically loaded wall panels, cracking of the AAC occurred on the inside face of the panel. The failures happened randomly at the top or bottom of the panel, and no apparent signs of longitudinal steel buckling were observed.

SUMMARY AND CONCLUSIONS

An experimental program was undertaken to study the structural behavior of three most common types of steel reinforced AAC elements. A total of 42 full-scale test specimens consisted of 12 floor panels, 24 lintels, and 6 vertical wall panels. All specimens were made of AAC Grade G2 and were supplied by manufacturers in the U.S. Floor panels and lintels were tested in bending, whereas the wall panels were tested under axial or eccentric compression loading. The ultimate load capacity, cracking behavior, deflection, and failure mode were observed and recorded for each test.

Information gained through this test program provides insight and better understanding of the structural behavior of reinforced AAC elements. The data obtained can also be used to verify proposed analytical methods for the design of AAC. Improved

design methodologies are needed to establish AAC as a reliable engineered construction material in the U.S.

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Table 1: AAC material grades defined by ASTM C 1386-98

AAC Grade	Nominal dry Density ^a pcf (Kg/m ³)	Density limits pcf (Kg/m ³)	Average compressive strength psi (MPa)	Minimum compressive strength psi (MPa)
G1	25 (400)	22 (350)-28 (450)	360 (2.5)	290 (2.0)
G2	31 (500)	28 (450)-34 (550)	360 (2.5)	290 (2.0)
G3	37 (600)	34 (550)-41 (650)	725 (5.0)	580 (4.0)

a. The dry density is the average density of the material after being dried in a ventilated oven at 100 °C until no further weight loss occurs with successive drying.

Table 2: Floor panels used in test program

Manu- facturer	Grade	Nominal f_{aac} psi (MPa)	Bottom long. steel reinf. ^a	Top long. steel reinf. ^a	Super- imposed design load psf (kN/m ²)	Test span ft (m)	Floor depth in. (mm)	Number of specimens
Ytong	G2 (31 pcf) (500 kg/m ³)	720 (5.0)	4 bars -	4 bars -	46 (2.2)	12.5 (3.8)	6 (150)	2
			7 mm dia.	7 mm dia.	39 (1.9)	16.5 (5.0)	8 (200)	2
			6 bars -	4 bars -	52 (2.5)	12.5 (3.8)	6 (150)	2
			7 mm dia.	7 mm dia.	48 (2.3)	16.5 (5.0)	8 (200)	2
			10 bars -	4 bars -	77 (3.7)	12.5 (3.8)	6 (150)	2
			7 mm dia.	7 mm dia.	73 (3.5)	16.5 (5.0)	8 (200)	2

a. ASTM A 82-97a, $f_y = 80$ ksi (550 MPa)

Table 3: Summary of Ytong AAC G2 floor panel results

Specimen I.D.	Bottom-mat reinforcement		P_{max}^* lb (kN)	P_{crack}^* lb (kN)	P_{crack}/P_{max} (%)	w_{max} psf (kN/m ²)	w_{crack} psf (kN/m ²)	SDL** psf (kN/m ²)	Safety factor
	Longitudinal	Cross							
1 YF 6-24-12.5 A	4 - 7mm	17 - 6mm	3858 (17.16)	1452 (6.46)	38	154 (7.4)	58 (2.8)	46	3.4
2 YF 6-24-12.5 A	4 - 7mm	17 - 6mm	3989 (17.74)	1190 (5.30)	30	160 (7.7)	48 (2.3)	(2.2)	3.5
1 YF 8-24-16.5 A	4 - 7mm	17 - 6mm	4235 (18.83)	1112 (4.95)	26	128 (6.1)	34 (1.6)	39	3.3
2 YF 8-24-16.5 A	4 - 7mm	17 - 6mm	4241 (18.86)	1159 (5.15)	27	129 (6.2)	35 (1.7)	(1.9)	3.3
1 YF 6-24-12.5 B	6 - 7mm	17 - 6mm	4551 (20.24)	1534 (6.82)	34	182 (8.7)	61 (2.9)	52	3.5
2 YF 6-24-12.5 B	6 - 7mm	17 - 6mm	4532 (20.16)	1608 (7.15)	35	181 (8.7)	64 (3.1)	(2.5)	3.5
1 YF 8-24-16.5 B	6 - 7mm	17 - 6mm	5809 (25.84)	1567 (6.97)	27	176 (8.4)	47 (2.3)	48	3.7
2 YF 8-24-16.5 B	6 - 7mm	17 - 6mm	6346 (28.23)	1741 (7.74)	27	192 (9.2)	53 (2.5)	(2.3)	4.0
1 YF 6-24-12.5 C	10 - 7mm	17 - 6mm	6118 (27.21)	2602 (11.57)	43	245 (11.7)	104 (5)	77	3.2
2 YF 6-24-12.5 C	10 - 7mm	17 - 6mm	5901 (26.25)	2539 (11.30)	43	236 (11.3)	102 (4.9)	(3.7)	3.1
1 YF 8-24-16.5 C	10 - 7mm	17 - 6mm	8082 (35.95)	2567 (11.41)	32	245 (11.7)	78 (3.7)	73	3.4
2 YF 8-24-16.5 C	10 - 7mm	17 - 6mm	7971 (35.46)	2262 (10.06)	28	242 (11.6)	69 (3.3)	(3.5)	3.3
1 YF 6-24-12.5 B: Down	6 - 7mm	17 - 6mm	3889 (17.30)	1061 (4.72)	27	156 (7.5)	42 (2.0)	-	-

* corrected loads: accounts for additional weight of device, not including weight of the panel
 (approximate weight of 6-inch panel = 17.5 psf (0.84 kN/m²), 8-inch panel = 23.3 psf (1.1 kN/m²))

** SDL = Ytong's superimposed design load

Table 4: Summary of Ytong AAC G2 floor panel deflections results

Specimen I.D.	Bottom-mat reinforcement		Stress at cracking psi (MPa)	Mid span deflection at first crack in. (mm)	Span / 300 in. (mm)
	Longitudinal	Cross			
1 YF 6-24-12.5 A	4 - 7mm	17 - 6mm	189 (1.3)	0.44 (11.1)	0.50 (12.7)
2 YF 6-24-12.5 A	4 - 7mm	17 - 6mm	155 (1.1)	0.33 (8.4)	0.50 (12.7)
1 YF 8-24-16.5 A	4 - 7mm	17 - 6mm	108 (0.7)	0.32 (8.1)	0.66 (16.7)
2 YF 8-24-16.5 A	4 - 7mm	17 - 6mm	112 (0.8)	0.28 (7.1)	0.66 (16.7)
1 YF 6-24-12.5 B	6 - 7mm	17 - 6mm	200 (1.4)	0.47 (11.9)	0.50 (12.7)
2 YF 6-24-12.5 B	6 - 7mm	17 - 6mm	209 (1.4)	0.47 (11.9)	0.50 (12.7)
1 YF 8-24-16.5 B	6 - 7mm	17 - 6mm	151 (1.0)	0.35 (8.9)	0.66 (16.7)
2 YF 8-24-16.5 B	6 - 7mm	17 - 6mm	168 (1.2)	0.42 (10.7)	0.66 (16.7)
1 YF 6-24-12.5 C	10 - 7mm	17 - 6mm	339 (2.3)	0.55 (14)	0.50 (12.7)
2 YF 6-24-12.5 C	10 - 7mm	17 - 6mm	331 (2.3)	0.54 (13.7)	0.50 (12.7)
1 YF 8-24-16.5 C	10 - 7mm	17 - 6mm	248 (1.7)	0.51 (13)	0.66 (16.7)
2 YF 8-24-16.5 C	10 - 7mm	17 - 6mm	219 (1.5)	0.44 (11.1)	0.66 (16.7)

Table 5: Lintels used in test program

Manu- facturer	Material grade	Nominal f_{ac} psi (MPa)	Bottom long. steel reinf.	Top long. steel reinf.	Test span ft (m)	Lintel width in. (mm)	Lintel depth in. (mm)	Number of specimens
Hebel	G2 (32 pcf) (510 kg/m ³)	720 (5.0)	4 - 8 mm dia.	2 - 8 mm dia.	3 (0.9)	8 (200)	8 (200)	2
			4 - 8 mm dia.	2 - 8 mm dia.	5 ^a (1.5)	8 (200)	8 (200)	2
			4 - 8 mm dia.	2 - 8 mm dia.	6 (1.8)	8 (200)	8 (200)	2
			4 - 8 mm dia.	2 - 8 mm dia.	3 (0.9)	10 (250)	8 (200)	2
			4 - 8 mm dia.	2 - 8 mm dia.	5 (1.5)	10 (250)	8 (200)	2
			4 - 8 mm dia.	2 - 8 mm dia.	7 (2.1)	10 (250)	8 (200)	2
Ytong	G2 (31 pcf) (500 kg/m ³)	720 (5.0)	4 - 7 mm dia.	4 - 7 mm dia.	3 (0.9)	8 (200)	8 (200)	2
			4 - 7 mm dia.	4 - 7 mm dia.	6 (1.8)	8 (200)	8 (200)	2
			4 - 7 mm dia.	4 - 7 mm dia.	6 (1.8)	8 (200)	16 (400)	2
			4 - 7 mm dia.	4 - 7 mm dia.	10 (3.0)	8 (200)	16 (400)	2
			8 - 7 mm dia.	8 - 7 mm dia.	10 (3.0)	8 (200)	24 (600)	2
			6 - 7 mm dia.	6 - 7 mm dia.	18 (5.5)	8 (200)	24 (600)	2

a. Additional lintel tested upside down.

Table 6: Summary of Hebel lintel test results

Specimen I.D.	U-Shaped Reinforcement		P_{max} lb (kN)	P_{crack} lb (kN)	P_{crack} / P_{max} (%)	w_{crack} psf (kN/m ²)	Stress at cracking psi (MPa)
	Longitudinal	Cross					
1 HL 8-8-3	6-8 mm dia	20-6 mm dia	22967 (102.16)	7221 (32.12)	31	3611 (172.9)	381 (2.6)
2 HL 8-8-3			16271 (72.38)	6877 (30.59)	42	3439 (164.7)	363 (2.5)
1 HL 8-8-5	6-8 mm dia	28-6 mm dia	12732 (56.63)	4500 (20.02)	35	1350 (64.6)	396 (2.7)
3 HL 8-8-5			11479 (51.06)	4314 (19.19)	38	1294 (62.0)	379 (2.6)
1 HL 8-8-6	6-8 mm dia	30-6 mm dia	10584 (47.08)	3900 (17.35)	37	975 (46.7)	411 (2.8)
2 HL 8-8-6			10102 (44.94)	3389 (15.08)	34	847 (40.6)	357 (2.5)
1 HL 10-8-3	6-8 mm dia	20-6 mm dia	17059 (75.88)	6322 (28.12)	37	2529 (121.1)	267 (1.8)
2 HL 10-8-3			23031 (102.45)	8885 (39.52)	39	3554 (170.2)	375 (2.6)
1 HL 10-8-5	6-8 mm dia	28-6 mm dia	10385 (46.19)	3924 (17.45)	38	942 (45.1)	276 (1.9)
2 HL 10-8-5			13832 (61.53)	4768 (21.21)	34	1144 (54.8)	335 (2.3)
1 HL 10-8-7	6-8 mm dia	36-6 mm dia	8178 (36.38)	2689 (11.96)	33	461 (22.1)	265 (1.8)
2 HL 10-8-7			10037 (44.65)	3102 (13.8)	31	532 (25.5)	305 (2.1)
2 HL 8-8-5 Down	6-8 mm dia	28-6 mm dia	8471 (37.68)	2735 (12.16)	32	820 (39.3)	-

Table 7: Summary of Ytong lintel test results

Specimen I.D.	Double-mat Reinforcement		P_{max} lb (kN)	P_{crack} lb (kN)	P_{crack} / P_{max} (%)	w_{crack} psf (kN/m ²)	Stress at cracking psi (MPa)
	Longitudinal	Cross					
1 YL 8-8-3	4-7 mm dia x2	16-6 mm dia x2	13045 (58.03)	3977 (17.69)	30	1989 (95.2)	210 (1.4)
2 YL 8-8-3			12127 (53.94)	2758 (12.27)	23	1379 (66.0)	145 (1.0)
1 YL 8-8-6	4-7 mm dia x2	22-6 mm dia x2	7730 (34.38)	1338 (5.95)	17	401 (19.2)	141 (1.0)
2 YL 8-8-6			7893 (35.11)	2025 (9.01)	26	607 (29.1)	214 (1.5)
1 YL 8-16-6	4-7 mm dia x2	23-6 mm dia x2	19055 (84.76)	5010 (22.29)	26	1253 (60.0)	132 (0.9)
2 YL 8-16-6			20352 (90.53)	4869 (21.66)	24	1217 (58.3)	128 (0.9)
1 YL 8-16-10	4-7 mm dia x2	30-6 mm dia x2	11502 (51.16)	3014 (13.41)	26	1206 (57.7)	132 (0.9)
2 YL 8-16-10			11504 (51.17)	4049 (18.01)	35	1620 (77.6)	178 (1.2)
1 YL 8-24-10	8-7 mm dia x2	36-6 mm dia x2	27002 (120.11)	7469 (33.22)	28	1792 (85.8)	146 (1.0)
2 YL 8-24-10			22490 (100.04)	9002 (40.04)	40	2160 (103.4)	176 (1.2)
1 YL 8-24-18	6-7 mm dia x2	48-6 mm dia x2	14271 (63.48)	3002 (13.35)	21	515 (24.7)	106 (0.7)
2 YL 8-24-18			16225 (72.17)	2424 (10.78)	15	416 (19.9)	85 (0.6)

Table 8: Vertical wall panels used in test program

Manu- facturer	Material grade	Nominal f'_{ac} psi (MPa)	Longitudinal steel reinforcement	Wall height ft (m)	Wall thickness in (mm)	Number of specimens	Axial loading type
Ytong	G2 (31 pcf) (500 kg/m ³)	720 (5.0)	4 - 6 mm dia.	8 (2.4)	6 (150)	2	concentric
			8 - 6 mm dia.	8 (2.4)	8 (200)	2	eccentric
			8 - 6 mm dia.	8 (2.4)	9.5 (240)	2	eccentric

Note: all specimens were 24 inches (600 mm) in width

Table 9: Vertical wall panel results

Specimen I.D.	P_{max} lb (kN)	Max. defl - inside (mm)	Gage Length in. (mm)	Max. strain - inside (in./in.)	Max. strain - outside (in./in.)	Stress σ_{max} psi (MPa)
1 YW 6-24-96	68625 (305.26)	0.253	64.375 (1635)	0.001405	0.001452	477 (3.3)
2 YW 6-24-96	62813 (279.41)	2.273	64.125 (1630)	0.001478	0.001276	436 (3.0)
1 YW 8-24-96	70063 (311.66)	2.255	64.500 (1638)	0.001661	0.000617	365 (2.5)
2 YW 8-24-96	78313 (348.35)	0.670	64.375 (1635)	0.001702	0.000599	408 (2.8)
1 YW 9.5-24-96	82438 (366.7)	1.706	64.375 (1635)	0.001754	0.000621	362 (2.5)
2 YW 9.5-24-96	82063 (365.03)	1.983	64.375 (1635)	0.001378	0.000854	360 (2.5)

Note: Eight control blocks were tested and yielded an average compressive strength of 480 psi (3.3 MPa)

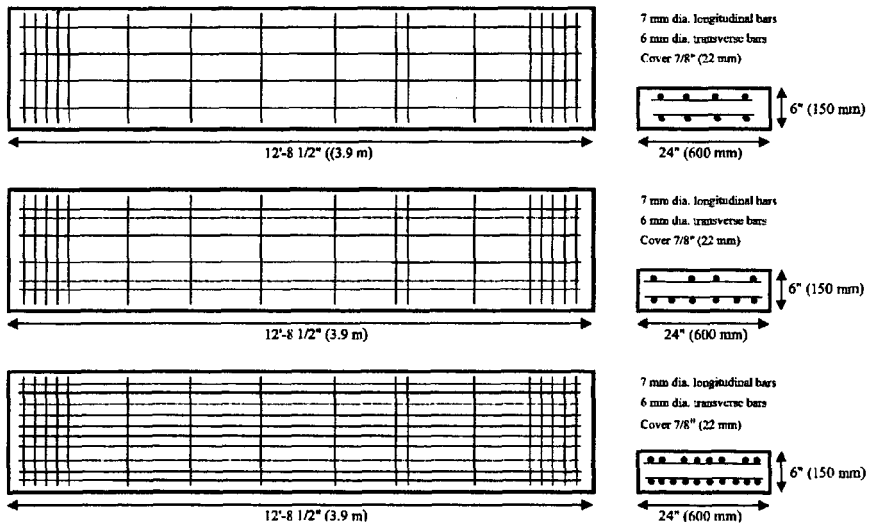


Figure 1: Reinforcement diagram for 6-inch floor panels tested at a 12.5-foot span