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
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 grounded sand, and lime. In some cases, it is acceptable to replace the finely grounded sand with fly ash. These materials are mixed with water and a small amount of aluminum power and cast into a mold. Hydrogen gas, a product of the reaction between the cement hydration products and the aluminum power, 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 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
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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 weak 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” 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.
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 0.05\(f'_{aac}\) 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.

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REFERENCE

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Table 1: ASTM C 1386 – 98 Physical Requirements

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

\(^a\) 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

200 x 200 x 600 mm
AAC Building Unit

B. Saw Cut: 2 Specimens Per Block

100 mm

Direction of Rise

Saw Cut

C. Tensile Stresses Induced Parallel to the Direction of Rise

100 mm

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

200 x 300 x 600 mm
AAC Building Unit
Direction of Rise

B. Saw Cuts

300 mm
Saw Cuts

C. 1 Specimen Per Block Third

300 mm
100 mm

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

A. Mini-Jumbo Block

200 x 300 x 600 mm
AAC Building Unit
Direction of Rise

B. Saw Cuts

300 mm
300 mm
Saw Cuts

C. 1 Specimen Per Block Half

100 mm
300 mm

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

\[ E_{AAC} = 6500 \ f'^{10.6}_{AAC} \]  \hspace{1cm} (Equation 1a)

\[ E_{AAC} = 887.7 \ f'^{10.6}_{AAC} \]  \hspace{1cm} (Equation 1b)

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

\[ f_{tAAC} = 2.4 \sqrt{f'_{AAC}} \]  \hspace{1cm} (Equation 2a)

\[ f_{tAAC} = 0.2 \sqrt{f'_{AAC}} \]  \hspace{1cm} (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}, $\phi_{cr}$) and the yielding points (M_y, $\phi_y$) 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 $\beta_1$ is taken as 0.67 for AAC and the minimum reinforcement of flexural members is calculated by

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

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

**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,

\[ V_{AAC} = 0.8 \sqrt{f'_{AAC}} b_w d \]  
\[ V_{AAC} = 0.07 \sqrt{f'_{AAC}} b_w d \]  
(Equation 6a)

(Equation 6b)

For members subject to axial compression,

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

\[ V_{AAC} = 0.07 \sqrt{f'_{AAC}} \left( 1 + \frac{N_u}{13.8 A_g} \right) b_w d \]  
(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

\[ V_s = \frac{A_v f_y d}{S} \]  
(Equation 8)

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

\[ V_{sb} = d_{long} f'_{AAC} \]  
(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,

\[ V_n = A_v f_y \mu \]  
(Equation 10)
In seismic loading combinations, the shear strength of AAC is calculated by either,

\[ V_n = N_u \mu, \]  

(Equation 11)

where \( N_u \) is the factored normal force at the considered shear plane, or the direct shear strength of the joint. \( \mu \) 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

\[ V_{AAC} = 0.86 \ell_w h \sqrt{f'_{AAC}} \sqrt{1 + \frac{N_u}{2.4 N_u}} \]  

(Equation 12a)

or

\[ V_{AAC} = 0.07 \ell_w h \sqrt{f'_{AAC}} \sqrt{1 + \frac{N_u}{0.2 N_u}} \]  

(Equation 12b)

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 \( \phi V_{AAC} \), the horizontal shear reinforcement shall be provided to satisfy Equation 4 and Equation 5, where the shear strength \( V_s \) is computed by

\[ V_s = \frac{A_v f_y d}{s_2}, \]  

(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.
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 $f'_{\text{AAC}}$ 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, $s_{\text{cross}}$, in factory installed reinforcement embedded in AAC is limited to less than or equal to the value required to satisfy Equation 15.

$$s_{\text{max}} = \frac{0.85 \cdot \frac{V_u}{\phi} \cdot d_{\text{cross}} \cdot f'_{\text{AAC}}}{1_{\text{cross}} \cdot \frac{V_u}{\phi}}$$ (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 $2s_{\text{min}}$.

$$n_{\text{cross,min}} = \frac{0.85 \cdot \frac{V_u}{\phi} \cdot d_{\text{cross}} \cdot f'_{\text{AAC}}}{1_{\text{cross}} \cdot \frac{V_u}{\phi} \cdot a}$$ (Equation 16)
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:
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 1.25fy.

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.

References


Notations

\( A_g \) = gross area of section, in\(^2\) (mm\(^2\))
\( A_i \) = 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)
\( d_{\text{cross}} \) = diameter of cross-wires, in. (mm)
\( d_{\text{long}} \) = diameter of longitudinal reinforcement, in. (mm)
\( E_{\text{AAC}} \) = modulus of elasticity of AAC, psi (MPa)
\( f'_{\text{AAC}} \) = specified compressive strength of AAC, psi (MPa)
\( f'_{\text{gr}} \) = specified compressive strength of grout, psi (MPa)
\( f_y \) = specified yield strength of nonprestressed reinforcement, psi (MPa)
\( f_{\text{AAC}} \) = modulus of rupture of AAC, psi (MPa)
\( f_t_{\text{AAC}} \) = 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)
\( l_{\text{cross}} \) = 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)
\( V_{\text{AAC}} \) = 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)
\( V_i \) = 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)
\( \rho_{1386} \) = air-dried density of AAC by ASTM C 1386, lb/ft\(^3\) (kg/m\(^3\))
\( w_{\text{strut}} \) = horizontal projection of the width of the compression strut, in. (m)

References

(1) ACI 523.5R – Guide for Using Autoclaved Aerated Concrete Panels
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
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 power 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 power, 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 1/5th to 1/3rd 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 a 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
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, \( P_{\text{max}} \) (or \( w_{\text{max}} \)), to the load at first crack, \( P_{\text{crack}} \) (or \( w_{\text{crack}} \)), 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, \( w_{\text{crack}} \), 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 \( w_{\text{crack}} \) 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.
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.
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, \( P_{\text{max}} = 65,720 \) 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.

ACKNOWLEDGMENTS

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REFERENCES


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

\(a\). 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

<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>Grade</th>
<th>p&lt;sub&gt;aac&lt;/sub&gt; psi (MPa)</th>
<th>Bottom long. steel reinf.&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Top long. steel reinf.&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Superimposed design load psf (kN/m&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>Test span ft (m)</th>
<th>Floor depth in. (mm)</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ytong</td>
<td>G2</td>
<td>720 (500 kg/m&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>4 bars - 7 mm dia. 4 bars - 7 mm dia.</td>
<td>46 (2.2) 39 (1.9)</td>
<td>12.5 (3.8) 16.5 (5.0)</td>
<td>6 (150) 8 (200)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6 bars - 7 mm dia. 4 bars - 7 mm dia.</td>
<td>52 (2.5) 48 (2.3)</td>
<td>12.5 (3.8) 16.5 (5.0)</td>
<td>6 (150) 8 (200)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10 bars - 7 mm dia. 4 bars - 7 mm dia.</td>
<td>77 (3.7) 73 (3.5)</td>
<td>12.5 (3.8) 16.5 (5.0)</td>
<td>6 (150) 8 (200)</td>
<td>2</td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup> ASTM A 82-97a, f_y = 80 ksi (550 MPa)

Table 3: Summary of Ytong AAC G2 floor panel results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Bottom-mat reinforcement</th>
<th>P&lt;sub&gt;max&lt;/sub&gt; * (lb)</th>
<th>P&lt;sub&gt;crack&lt;/sub&gt; * (lb)</th>
<th>P&lt;sub&gt;crack&lt;/sub&gt; /P&lt;sub&gt;max&lt;/sub&gt; (%)</th>
<th>w&lt;sub&gt;max&lt;/sub&gt; psf (kN/m&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>w&lt;sub&gt;crack&lt;/sub&gt; psf (kN/m&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>SDL** psf (kN/m&lt;sup&gt;2&lt;/sup&gt;)</th>
<th>Safety factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 YF 6-24-12.5 A</td>
<td>4 - 7mm 17 - 6mm</td>
<td>3858 (17.16)</td>
<td>1452 (6.46)</td>
<td>38</td>
<td>154 (7.4)</td>
<td>160 (7.7)</td>
<td>58 (2.8)</td>
<td>46 (2.2)</td>
</tr>
<tr>
<td>2 YF 6-24-12.5 A</td>
<td>4 - 7mm 17 - 6mm</td>
<td>3989 (17.74)</td>
<td>1190 (5.30)</td>
<td>30</td>
<td>128 (6.1)</td>
<td>129 (6.2)</td>
<td>34 (1.6)</td>
<td>39 (1.9)</td>
</tr>
<tr>
<td>1 YF 8-24-16.5 A</td>
<td>4 - 7mm 17 - 6mm</td>
<td>4235 (18.83)</td>
<td>1112 (4.95)</td>
<td>26</td>
<td>182 (8.7)</td>
<td>181 (8.7)</td>
<td>61 (2.9)</td>
<td>52 (2.5)</td>
</tr>
<tr>
<td>2 YF 8-24-16.5 A</td>
<td>4 - 7mm 17 - 6mm</td>
<td>4241 (18.86)</td>
<td>1159 (5.15)</td>
<td>27</td>
<td>128 (6.1)</td>
<td>129 (6.2)</td>
<td>35 (1.7)</td>
<td>39 (1.9)</td>
</tr>
<tr>
<td>1 YF 6-24-12.5 B</td>
<td>6 - 7mm 17 - 6mm</td>
<td>4551 (20.24)</td>
<td>1534 (6.82)</td>
<td>34</td>
<td>182 (8.7)</td>
<td>181 (8.7)</td>
<td>64 (3.1)</td>
<td>52 (2.5)</td>
</tr>
<tr>
<td>2 YF 6-24-12.5 B</td>
<td>6 - 7mm 17 - 6mm</td>
<td>4532 (20.16)</td>
<td>1608 (7.15)</td>
<td>35</td>
<td>182 (8.7)</td>
<td>181 (8.7)</td>
<td>64 (3.1)</td>
<td>52 (2.5)</td>
</tr>
<tr>
<td>1 YF 8-24-16.5 B</td>
<td>6 - 7mm 17 - 6mm</td>
<td>5089 (25.84)</td>
<td>1567 (6.97)</td>
<td>27</td>
<td>176 (8.4)</td>
<td>192 (9.2)</td>
<td>47 (2.3)</td>
<td>48 (2.3)</td>
</tr>
<tr>
<td>2 YF 8-24-16.5 B</td>
<td>6 - 7mm 17 - 6mm</td>
<td>6346 (28.23)</td>
<td>1741 (7.74)</td>
<td>27</td>
<td>176 (8.4)</td>
<td>192 (9.2)</td>
<td>53 (2.5)</td>
<td>48 (2.3)</td>
</tr>
<tr>
<td>1 YF 6-24-12.5 C</td>
<td>10 - 7mm 17 - 6mm</td>
<td>6118 (22.21)</td>
<td>2602 (11.57)</td>
<td>43</td>
<td>245 (11.7)</td>
<td>236 (11.3)</td>
<td>104 (5)</td>
<td>77 (3.7)</td>
</tr>
<tr>
<td>2 YF 6-24-12.5 C</td>
<td>10 - 7mm 17 - 6mm</td>
<td>5901 (26.25)</td>
<td>2539 (11.30)</td>
<td>43</td>
<td>245 (11.7)</td>
<td>236 (11.3)</td>
<td>102 (4.9)</td>
<td>77 (3.7)</td>
</tr>
<tr>
<td>1 YF 8-24-16.5 C</td>
<td>10 - 7mm 17 - 6mm</td>
<td>8082 (35.95)</td>
<td>2567 (11.41)</td>
<td>32</td>
<td>245 (11.7)</td>
<td>242 (11.6)</td>
<td>78 (3.7)</td>
<td>73 (3.5)</td>
</tr>
<tr>
<td>2 YF 8-24-16.5 C</td>
<td>10 - 7mm 17 - 6mm</td>
<td>7971 (35.46)</td>
<td>2262 (10.06)</td>
<td>28</td>
<td>245 (11.7)</td>
<td>242 (11.6)</td>
<td>69 (3.3)</td>
<td>73 (3.5)</td>
</tr>
<tr>
<td>1 YF 6-24-12.5 B</td>
<td>Down - 6 - 7mm 17 - 6mm</td>
<td>3889 (17.30)</td>
<td>1061 (4.72)</td>
<td>27</td>
<td>156 (7.5)</td>
<td>42 (2.0)</td>
<td>-</td>
<td>-</td>
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</tbody>
</table>

* 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/m<sup>2</sup>), 8-inch panel = 23.3 psf (1.1 kN/m<sup>2</sup>))

** SDL = Ytong’s superimposed design load
### Table 4: Summary of Ytong AAC G2 floor panel deflections results

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

### Table 5: Lintels used in test program

<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>Material grade</th>
<th>Nominal $f_{\text{ac}}$ psi (MPa)</th>
<th>Bottom long. steel reinf.</th>
<th>Top long. steel reinf.</th>
<th>Test span ft (m)</th>
<th>Lintel width in. (mm)</th>
<th>Lintel depth in. (mm)</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hebel</td>
<td>G2 (32 pcf)</td>
<td>720 (5.0)</td>
<td>4 - 8 mm dia.</td>
<td>2 - 8 mm dia.</td>
<td>3 (0.9)</td>
<td>8 (200)</td>
<td>8 (200)</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>(310 kg/m³)</td>
<td></td>
<td>4 - 8 mm dia.</td>
<td>2 - 8 mm dia.</td>
<td>5 (1.5)</td>
<td>8 (200)</td>
<td>8 (200)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 - 8 mm dia.</td>
<td>2 - 8 mm dia.</td>
<td>6 (1.8)</td>
<td>8 (200)</td>
<td>8 (200)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 - 8 mm dia.</td>
<td>2 - 8 mm dia.</td>
<td>3 (0.9)</td>
<td>10 (250)</td>
<td>8 (200)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 - 8 mm dia.</td>
<td>2 - 8 mm dia.</td>
<td>5 (1.5)</td>
<td>10 (250)</td>
<td>8 (200)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 - 8 mm dia.</td>
<td>2 - 8 mm dia.</td>
<td>7 (2.1)</td>
<td>10 (250)</td>
<td>8 (200)</td>
<td></td>
</tr>
<tr>
<td>Ytong</td>
<td>G2 (31 pcf)</td>
<td>720 (5.0)</td>
<td>4 - 7 mm dia.</td>
<td>4 - 7 mm dia.</td>
<td>3 (0.9)</td>
<td>8 (200)</td>
<td>8 (200)</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>(500 kg/m³)</td>
<td></td>
<td>4 - 7 mm dia.</td>
<td>4 - 7 mm dia.</td>
<td>6 (1.8)</td>
<td>8 (200)</td>
<td>8 (200)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 - 7 mm dia.</td>
<td>4 - 7 mm dia.</td>
<td>6 (1.8)</td>
<td>8 (200)</td>
<td>16 (400)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8 - 7 mm dia.</td>
<td>4 - 7 mm dia.</td>
<td>10 (3.0)</td>
<td>8 (200)</td>
<td>16 (400)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6 - 7 mm dia.</td>
<td>6 - 7 mm dia.</td>
<td>18 (5.5)</td>
<td>8 (200)</td>
<td>24 (600)</td>
<td></td>
</tr>
</tbody>
</table>

a. Additional lintel tested upside down.
### Table 6: Summary of Hebel lintel test results

<table>
<thead>
<tr>
<th>Specimen ID.</th>
<th>U-Shaped Reinforcement</th>
<th>( P_{\text{max}} ) (lb)</th>
<th>( P_{\text{crack}} ) (lb)</th>
<th>( \frac{P_{\text{crack}}}{P_{\text{max}}} ) (%)</th>
<th>( W_{\text{crack}} ) (psf)</th>
<th>( W_{\text{crack}} ) (psi)</th>
<th>Stress at cracking psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 HL 8-8-3</td>
<td>6-8 mm dia, 20-6 mm dia</td>
<td>22967 (102.16)</td>
<td>7221 (32.12)</td>
<td>31 (172.9)</td>
<td>3611</td>
<td>3349</td>
<td>3439 (164.7)</td>
</tr>
<tr>
<td>2 HL 8-8-3</td>
<td>6-8 mm dia, 28-6 mm dia</td>
<td>12732 (56.63)</td>
<td>4500 (20.02)</td>
<td>35 (64.6)</td>
<td>1350</td>
<td>1294</td>
<td>1294 (62.0)</td>
</tr>
<tr>
<td>3 HL 8-8-5</td>
<td>6-8 mm dia, 30-6 mm dia</td>
<td>10584 (47.08)</td>
<td>3900 (17.35)</td>
<td>37 (46.7)</td>
<td>975</td>
<td>847</td>
<td>847 (40.6)</td>
</tr>
<tr>
<td>2 HL 8-8-6</td>
<td>6-8 mm dia, 20-6 mm dia</td>
<td>17059 (75.88)</td>
<td>6322 (28.12)</td>
<td>37 (121.1)</td>
<td>2529</td>
<td>2354</td>
<td>2354 (110.2)</td>
</tr>
<tr>
<td>1 HL 10-8-3</td>
<td>6-8 mm dia, 28-6 mm dia</td>
<td>13045 (58.03)</td>
<td>3977 (17.69)</td>
<td>30 (95.2)</td>
<td>1989</td>
<td>1739</td>
<td>1739 (85.5)</td>
</tr>
<tr>
<td>2 HL 10-8-3</td>
<td>6-8 mm dia, 4-7 mm dia</td>
<td>7730 (34.38)</td>
<td>2258 (9.55)</td>
<td>23 (34.3)</td>
<td>401</td>
<td>360</td>
<td>360 (17.2)</td>
</tr>
<tr>
<td>1 HL 10-8-5</td>
<td>6-8 mm dia, 22-6 mm dia</td>
<td>19055 (84.76)</td>
<td>5010 (22.29)</td>
<td>26 (60.0)</td>
<td>1253</td>
<td>1144</td>
<td>1144 (54.8)</td>
</tr>
<tr>
<td>2 HL 10-8-7</td>
<td>6-8 mm dia, 23-6 mm dia</td>
<td>11502 (51.16)</td>
<td>3014 (13.41)</td>
<td>30 (57.7)</td>
<td>1206</td>
<td>1127</td>
<td>1127 (54.8)</td>
</tr>
<tr>
<td>1 HL 10-8-7</td>
<td>6-8 mm dia, 36-6 mm dia</td>
<td>11504 (51.17)</td>
<td>4049 (18.01)</td>
<td>35 (77.6)</td>
<td>1620</td>
<td>1560</td>
<td>1560 (75.5)</td>
</tr>
<tr>
<td>2 HL 8-8-5</td>
<td>6-8 mm dia, 28-6 mm dia</td>
<td>847 (37.68)</td>
<td>2735 (12.16)</td>
<td>32 (39.3)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

### Table 7: Summary of Ytong lintel test results

<table>
<thead>
<tr>
<th>Specimen ID.</th>
<th>Double-mat Reinforcement</th>
<th>( P_{\text{max}} ) (kN)</th>
<th>( P_{\text{crack}} ) (kN)</th>
<th>( \frac{P_{\text{crack}}}{P_{\text{max}}} ) (%)</th>
<th>( W_{\text{crack}} ) (kN/m²)</th>
<th>( W_{\text{crack}} ) (psi)</th>
<th>Stress at cracking psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 YL 8-8-3</td>
<td>4-7 mm dia x2, 16-6 mm dia x2</td>
<td>13045 (58.03)</td>
<td>3977 (17.69)</td>
<td>30 (95.2)</td>
<td>1989</td>
<td>1739</td>
<td>1739 (85.5)</td>
</tr>
<tr>
<td>2 YL 8-8-3</td>
<td>4-7 mm dia x2, 22-6 mm dia x2</td>
<td>7730 (34.38)</td>
<td>2258 (9.55)</td>
<td>23 (34.3)</td>
<td>401</td>
<td>360</td>
<td>360 (17.2)</td>
</tr>
<tr>
<td>1 YL 8-8-6</td>
<td>4-7 mm dia x2, 23-6 mm dia x2</td>
<td>19055 (84.76)</td>
<td>5010 (22.29)</td>
<td>30 (60.0)</td>
<td>1253</td>
<td>1144</td>
<td>1144 (54.8)</td>
</tr>
<tr>
<td>2 YL 8-8-6</td>
<td>4-7 mm dia x2, 30-6 mm dia x2</td>
<td>11502 (51.16)</td>
<td>3014 (13.41)</td>
<td>35 (77.6)</td>
<td>1620</td>
<td>1560</td>
<td>1560 (75.5)</td>
</tr>
<tr>
<td>1 YL 8-16-6</td>
<td>4-7 mm dia x2, 36-6 mm dia x2</td>
<td>11504 (51.17)</td>
<td>4049 (18.01)</td>
<td>30 (77.7)</td>
<td>1620</td>
<td>1560</td>
<td>1560 (75.5)</td>
</tr>
<tr>
<td>2 YL 8-16-6</td>
<td>4-7 mm dia x2, 48-6 mm dia x2</td>
<td>14271 (63.48)</td>
<td>3002 (13.35)</td>
<td>35 (77.7)</td>
<td>1620</td>
<td>1560</td>
<td>1560 (75.5)</td>
</tr>
<tr>
<td>1 YL 8-24-10</td>
<td>8-7 mm dia x2, 36-6 mm dia x2</td>
<td>27002 (120.11)</td>
<td>7469 (33.22)</td>
<td>28 (85.8)</td>
<td>1792</td>
<td>1692</td>
<td>1692 (84.0)</td>
</tr>
<tr>
<td>2 YL 8-24-10</td>
<td>6-7 mm dia x2, 48-6 mm dia x2</td>
<td>16225 (72.17)</td>
<td>2424 (10.78)</td>
<td>40 (21.4)</td>
<td>1792</td>
<td>1692</td>
<td>1692 (84.0)</td>
</tr>
<tr>
<td>1 YL 8-24-18</td>
<td>6-7 mm dia x2, 48-6 mm dia x2</td>
<td>14271 (63.48)</td>
<td>3002 (13.35)</td>
<td>21 (24.7)</td>
<td>515</td>
<td>416</td>
<td>416 (19.9)</td>
</tr>
<tr>
<td>2 YL 8-24-18</td>
<td>6-7 mm dia x2, 48-6 mm dia x2</td>
<td>16225 (72.17)</td>
<td>2424 (10.78)</td>
<td>15 (19.9)</td>
<td>85 (0.6)</td>
<td>85 (0.6)</td>
<td>85 (0.6)</td>
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</table>
Table 8: Vertical wall panels used in test program

<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>Material grade</th>
<th>Nominal f_{sec} (ksi)</th>
<th>Longitudinal steel reinforcement</th>
<th>Wall height (ft)</th>
<th>Wall thickness (in.)</th>
<th>Number of specimens</th>
<th>Axial loading type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ytong</td>
<td>G2 (31 psf)</td>
<td>720 (5.0)</td>
<td>4-6 mm dia.</td>
<td>8 (2.4)</td>
<td>6 (150)</td>
<td>2</td>
<td>concentric</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>eccentric</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8-6 mm dia.</td>
<td>8 (2.4)</td>
<td>8 (200)</td>
<td>2</td>
<td>eccentric</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8-6 mm dia.</td>
<td>8 (2.4)</td>
<td>9.5 (240)</td>
<td>2</td>
<td>eccentric</td>
</tr>
</tbody>
</table>

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

Table 9: Vertical wall panel results

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

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.
Figure 2: AAC floor panel test setup

Figure 3: Reinforcement diagram for Hebel 8 by 8-inch lintels

Figure 4: AAC lintel test setup
Autoclaved Aerated Concrete

Wall Panel: 6" x 24" x 8'-6" (150 x 600 x 2440 mm)

4-6 mm diameter longitudinal bars centered in panel
10-5 mm diameter transverse bars
cover = 1 3/4" (45 mm)

Figure 5: Concentrically loaded wall panel reinforcement scheme

Wall Panel: 8" x 24" x 8'-6" (200 x 600 x 2440 mm)
and 9.5" x 24" x 8'-6" (241 x 600 x 2440 mm)

4-6 mm diameter longitudinal bars in each face
20-5 mm diameter transverse bars
cover = 1 3/4" (45 mm)

Figure 6: Eccentrically loaded wall panel reinforcement scheme
Figure 7: AAC wall panel test setup
Technical Justification for Proposed Design Provisions for AAC Structures: Introduction and Shear Wall Tests

by R. E. Klingner, J. E. Tanner, J. L. Varela, M. Brightman, J. Argudo, and U. Cancino

Synopsis: This paper summarizes the initial phases of the technical justification for proposed design provisions for AAC structures in the US. It is divided into two parts.

The first part gives general background information, and presents an overall design strategy. Autoclaved aerated concrete (AAC), a lightweight cementitious material originally developed in Europe more than 70 years ago and now widely used around the world, has recently been introduced into the US construction market. AAC elements can contain conventional reinforcement in grouted cores, either alone or with factory-installed reinforcement. To facilitate the use of AAC in the US market, an integrated seismic-qualification program has been carried out, involving general seismic design provisions, specific element design provisions, and material specifications.

The second part describes the design and testing of a suite of 14 AAC shear wall specimens, with aspect ratios from 0.6 to 3, under in-plane reversed cyclic loads at the University of Texas at Austin. The results of these tests have been used to develop predictive models and reliable design equations for AAC shear walls, the primary lateral force-resisting element of AAC structural systems.

Keywords: autoclaved aerated concrete; cellular concrete; design; earthquake
PART 1: OVERVIEW OF CODE-DEVELOPMENT STRATEGY FOR AAC

Introduction to Autoclaved Aerated Concrete (AAC)

Autoclaved Aerated Concrete (AAC) 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 (1). 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 cellular structure (Figure 1). Material specifications for this product are prescribed in ASTM C 1386 [1].

In Table 1, 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 of conventional concrete, and about the same ratio of compressive strength, making it potentially suitable for cladding and infills, and for bearing-wall components of low- to medium-rise structures. Its thermal conductivity is one-sixth or less that of concrete, making it potentially energy-efficient. Its fire rating is slightly longer than that of conventional concrete of the same thickness, making it potentially useful in applications where fire resistance is important. Because of its internal porosity, AAC has very low sound transmission, making it potentially useful acoustically.
Autoclaved Aerated Concrete

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

In the US, modern uses of AAC began in 1990, for residential and commercial projects in the southeastern states. US production of plain and reinforced AAC started in 1995 in the southeast, and has since spread to other parts of the country. AAC products include masonry-type units, reinforced panels, and specialty elements such as lintels, floor or roof planks, and stairs (Figure 2).

AAC elements can be produced with dimensional tolerances as small as 1/16 in. (1.5 mm). As a consequence, AAC masonry units can be laid with mortar joints approximately 3/8 in. (9 mm) thick, and also with thinner joints. The exterior face of the resulting AAC masonry wall is then protected from the elements using an exterior wythe of masonry, a cladding system, or a breathable coating resistant to penetration by liquid water. The interior face can be plastered, furred, or painted.

AAC masonry units are laid atop a leveling bed of ASTM C270 mortar, and are shimmed if necessary to achieve a bed course that is plumb, level and true (Figure 3). Most AAC masonry is “thin-bed masonry,” laid with joints about 1.5 mm thick. Subsequent courses of thin-bed masonry are laid using special thin-bed mortar, using a special notched trowel available from the AAC manufacturer. Units are laid in alignment with either the inside or the outside plane of the wall. Minor adjustments can be made by sanding the exposed faces of the units with a sanding board.

AAC masonry units themselves are unreinforced. Field reinforcement can be installed horizontally in lintel units, or vertically in grouted cells 3 in. or 4 in. in diameter. AAC masonry to be grouted is wetted thoroughly before grouting, to ensure that the grout flows to completely fill the space to be grouted. A small-diameter vibrator is inserted in the cell to be grouted; the grout is poured in; and the vibrator is withdrawn, consolidating the grout.

Mortar for thin-bed AAC masonry is a polymer-fortified mixture of portland cement and fine sand, produced by mortar manufacturers to meet performance standards approved by AAC manufacturers. In general, AAC manufacturers have approved lists of AAC mortar suppliers, based on internal performance criteria for AAC mortar. Those criteria generally 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.
Research on AAC

Because it has been used extensively in Europe for more than 70 years, AAC has been extensively researched there. Extensive manufacturer testing is available world-wide. In the US, basic evaluation-service testing has been carried out on structural performance, fire resistance, and thermal and acoustical properties. In the US, structural testing has been conducted at the University of Texas at Arlington (by Ytong); at Clemson University (by Ytong); and at the University of Alabama at Birmingham (by Hebel, subsequently known as Matrix, and now known as Babb). That testing verified compressive strength, tensile strength, diagonal tensile strength, modulus of rupture, and modulus of elasticity. Results have been circulated internally to groups developing design provisions, but have not been published. Most recently, extensive testing of shear walls and a two-story assemblage has been completed at The University of Texas at Austin [2-5]. Open publication is an explicit objective of that recent research, described in more detail here.

Seismic qualification of AAC masonry is based in general on experience in the Middle East and Japan, and on an extensive analytical and experimental research program being conducted at The University of Texas at Austin. That seismic qualification program is intended to develop design models, draft code provisions, analytical models, and R and Cd factors. It involved confirmatory material tests, more than 15 reversed cyclic load tests on different configurations of AAC shear walls, and finally proof tests of a full-scale, two-story AAC assemblage under quasi-static, reversed cyclic loading. That seismic research is the topic of several other papers at this conference, and is not discussed further here.

Possible Approval Approaches for AAC Masonry

At the present time, proposed AAC masonry buildings in the US are approved on a case-by-case basis. Product approvals are obtained through evaluation services of model code agencies; designs are carried out in accordance with industry guidelines; and project approvals are obtained through local building officials.

This approval approach, while feasible on a limited scale, is cumbersome and expensive. It would be better to have AAC masonry buildings approved on a national basis. AAC material would be addressed by ASTM material specifications; design provisions, developed under ANSI consensus procedures, would be referenced by legally adopted building codes; and project approvals would be obtained automatically.
How to Achieve the Goal of a National Basis for Approval of AAC Masonry Buildings

To achieve the goal of a national basis for approval of AAC masonry buildings, the following steps are required:

1. Evaluate the existing and probable future code framework for masonry and for concrete, and determine where within that framework should design provisions be developed for AAC masonry and for reinforced AAC panels, and where within that framework should material specification specifications be developed.

2. Using a comprehensive, integrated research program, synthesizing existing research and conducting new research when necessary, develop the performance data necessary to justify a proposed design approach for AAC masonry, including seismic design. For seismic design, carry out the experimental and analytical research necessary to propose and justify $R$ and $C_d$ factors. The research program would be specifically set up to deliver draft code provisions, commentary, and technical justification.

3. Prepare a draft set of code provisions, code commentary, and code “super-commentary” (extensive technical justification for proposed provisions and commentary), for design of AAC masonry.

4. Prepare a draft set of code provisions, code commentary, and code “super-commentary” (extensive technical justification for proposed provisions and commentary), for design using reinforced AAC panels. The design provisions for AAC masonry and reinforced AAC panels, while possibly different in format, should produce similar results.

5. Prepare a draft set of ASTM specifications for AAC masonry, and for other aspects of AAC construction.


That overall code framework is shown schematically in Figure 5, and is discussed in detail in the rest of this paper.

Proposed Integrated Design Framework for AAC Structures

Review of Code Development Process in the US -- In the US, structural design provisions are developed under ANSI consensus procedures, and the resulting documents are referenced by model codes. Over the past 10 years, model-code harmonization has resulted in the development of a predominant harmonized model code, the International Building Code (IBC), whose first issue was the 2000 IBC [6], and which will be updated at 3-year intervals. Another proposed model code is being developed by the National
Proposed Design Provisions for AAC Masonry -- In the US, development of masonry design provisions by an ANSI consensus process is the responsibility of the Masonry Standards Joint Committee (MSJC), sponsored by the American Concrete Institute (ACI), the American Society of Civil Engineers (ASCE), and The Masonry Society (TMS). Beginning with its 2003 edition, the IBC will essentially reference the MSJC provisions.

The MSJC design provisions, whose latest version is the 2002 MSJC Code and Specification [8, 9], cover a wide variety of design approaches (strength, allowable-stress, empirical) and materials (clay, concrete, glass block). Based on the combination of test results from The University of Texas at Austin, the University of Alabama at Birmingham, and elsewhere, a proposed design approach was developed for AAC masonry, with the following characteristics:

- A strength approach, consistent with Chapter 3 of the 2002 MSJC provisions. Strength provisions are preferred over allowable-stress provisions because they offer a more consistent factor of safety against structural failure; because they offer a better opportunity for harmonization with design provisions for reinforced panels; and connote reliability at the level of the material and the structural system.

- Design provisions for flexure, shear and anchorage that are generally similar to current strength-design provisions for other types of masonry, and that produce final designs similar to those produced by the proposed ACI provisions for reinforced AAC elements.

The first set of proposed design provisions, commentary, and “super-commentary” was introduced to the AAC Masonry Subcommittee of the MSJC early in 2002. Since then, it has been refined in response to MSJC Main Committee ballot comments and additional research results. Additional refinement within the MSJC process is expected to result in the passage of successful provisions within the 2005 code cycle.

Proposed Design Provisions for Reinforced AAC Panels -- In the US, development of design provisions for reinforced concrete under the ANSI consensus process is the responsibility of ACI Committee 318. The 2000 IBC essentially references ACI 318-99; the 2003 IBC is expected to reference ACI 318-02 [10]; and this is expected to continue for future IBC cycles.

The design provisions of ACI 318 address the strength design of a wide variety of conventional reinforced concrete elements similar to AAC applications, including
prefabricated wall panels. Based on the combination of test results from The University of Texas at Austin, the University of Alabama at Birmingham, and elsewhere, a proposed design approach was developed for reinforced AAC elements, with the following characteristics:

- A strength approach, consistent with ACI 318-02.
- Design provisions for flexure, shear and anchorage that are generally similar to current ACI 318 strength-design provisions for reinforced concrete elements, and that produce final designs similar to those produced by the proposed MSJC provisions for AAC masonry.

The first set of proposed design provisions, commentary, and “super-commentary” was introduced to ACI Subcommittee 523A (Autoclaved Aerated Cellular Concrete) in the fall of 2002. Because ACI 523A is a relatively new subcommittee, the design provisions, commentary, and “super-commentary” were introduced as appendices to a non-mandatory design guide on AAC. It is anticipated that the guide will be refined in response to balloting within Subcommittee 523A and Committee 523. After it has been approved by Committee 523, it will be offered to ACI 318 as a basis for mandatory-language design provisions and commentary, as an appendix to ACI 318. Because these provisions must be discussed and refined within ACI Committee 318 as well as ACI 523, their timetable for approval will probably be extended longer than for their counterpart provisions for AAC masonry. The authors believe that a reasonable goal would be approval within the 2008 ACI 318 cycle.

**Proposed R and C<sub>d</sub> Factors for AAC Structures** -- Because seismic design is an important aspect of the proposed design provisions for AAC structures, whether of masonry units or reinforced panels, it is necessary to develop R and C<sub>d</sub> factors for use with ASCE 7, the seismic load document referenced by model codes such as the 2000 IBC.

The seismic force-reduction factor (R) specified in seismic design codes is intended to account for energy dissipation through inelastic structural deformation (ductility), and structural over-strength. The factor (R) is based on observation of the performance of different structural systems in previous strong earthquakes, on technical justification, and on tradition.

For structures of autoclaved aerated concrete (AAC), the force-reduction factor (R) and the corresponding displacement-amplification factor (C<sub>d</sub>) must be based on laboratory test results and numerical simulation of the response of AAC structures subjected to earthquake ground motions. The proposed factors must then be verified against the observed response of AAC structures in strong earthquakes.

Values of R and C<sub>d</sub> for AAC structures are being proposed in two code-development arenas.
In the Fall of 2002, values are being proposed to ICBO ES (a model-code evaluation service), as part of a proposed ICBO ES listing for AAC structural components and systems. That listing is intended to facilitate the use of such systems on a nationwide basis, until the consensus design provisions proposed above are incorporated in MSJC and ACI documents, and referenced by model codes.

After approval by ICBO ES, the same values will be proposed for adoption by ASCE 7. When adopted by ASCE 7, they will form a part of the design package for AAC structural elements.

**Proposed ASTM Specifications for AAC Construction** -- ASTM traditionally deals with specifications for materials and methods of test. For the past several years, standards-development work regarding AAC has been going on in two committees:

**In 1998**, ASTM Subcommittee C-27.60 (Precast Concrete Elements of AAC) developed a material standard for AAC: C 1386-98 (Standard Specification for Precast Autoclaved Aerated Concrete Wall Units). Subcommittee C27-60 has also developed a standard for reinforced AAC panels: C 1452-00 (Standard Specification for Reinforced Autoclaved Aerated Concrete Units). That subcommittee is also working on a standard method of test for determining the modulus of AAC.

**Since 1998**, ASTM Subcommittee C-15.10 (Autoclaved Aerated Concrete Masonry) has been developing a standard practice for autoclaved aerated concrete masonry. It is hoped that work on that standard will be completed soon. It references the AAC material provisions of ASTM C 1386-98, and also contains construction provisions. When design provisions for AAC masonry are incorporated into the MSJC Code, it is probable that the construction provisions will be incorporated into the MSJC Specification.

**Summary of Part 1**

Autoclaved aerated concrete (AAC), a lightweight cementitious material originally developed in Europe more than 70 years ago and now widely used around the world, has recently been introduced into the US construction market. In this paper, code-development efforts in the US related to AAC are summarized, with emphasis on AAC masonry.

Using coordinated research at The University of Texas at Austin, the University of Alabama at Birmingham, and elsewhere, integrated design provisions, commentary, and “super-commentary” (extensive technical justification) have been developed for AAC masonry (including field reinforcement), and for factory-reinforced AAC panels. Masonry design provisions are being studied and refined within the Masonry Standards Joint Committee; reinforced-panel provisions are being studied and refined within ACI Subcommittee 523A; values for the seismic-design factors $R$ and $C_d$ are being developed
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for use by ICBO ES and ASCT 7; and material specifications and standard practices have been or are being developed by ASTM Subcommittee C-15.10 and C-27.60.

An innovative aspect of this code-development strategy is the planning of research deliverables to include specific draft design provisions, commentary, and supporting “super-commentary.” That aspect could usefully be applied to other materials as well.

PART 2: DESIGN AND TESTING OF AAC SHEAR WALLS

Introduction

A suite of 14 Autoclaved Aerated Concrete (AAC) shear wall specimens, with aspect ratios (height of the point of load application divided by the plan length) from 0.6 to 3, has been tested at the University of Texas at Austin [2-4]. The shear walls were designed to be either shear- or flexure-dominated. The shear-dominated walls were heavily reinforced in flexure using external reinforcement. The flexure-dominated walls had light longitudinal reinforcement. The test setup is shown in Figure 6, and details of each specimen are presented in Table 2. In that table, the number after the supplier’s name identifies a particular shipment of AAC material.

In-plane reversed cyclic load was applied to the specimens at UT Austin through hydraulic rams connected to a reaction wall. The rams were attached to the specimen through a reinforced concrete loading beam connected to the wall with a Type S leveling bed. The walls were loaded axially by a combination of hydraulic rams, and post-tensioned external rods. As the specimens displaced laterally in-plane, they rotated about the compression toe; tensile forces increased in the rods on the tension side, and decreased in the rods on the compression side. The net axial force in the UT Austin walls remained approximately constant under reverse cyclic loading by maintaining a constant load in the hydraulic rams through a mechanical “load maintainer” and by ensuring some post-tensioning force remained in the external rods. A complete description of the test set-up and results is presented in References 2, 3 and 4. Information on synthesis of data on material tests from UT Austin, the University of Alabama at Birmingham and other laboratories is presented in Reference 5. Information is currently being prepared on final shear-wall tests conducted at UT Austin.1

Additional information was obtained from a suite of 12 shear-wall tests performed by Hebel (Germany)2. Each of those walls measured 8.2 ft (2.5 m) long, 8.2 ft (2.5 m) tall and 9.5 in. (0.24 m) thick, for an aspect ratio of 1.0. All were constructed of modular block in one-fifth or one-half running bond. Three Hebel specimens used mortared head joints, and were laid in one-fifth running bond. The remaining nine Hebel specimens did not use mortar in the head joints.

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1 MS Thesis in preparation, Ulises Cancino, Dept. of Civil Engineering, The University of Texas at Austin, August 2003.
2 Personal communication, Violandi Vratsanou, Hebel AG, Germany, November 2000
The specimens tested by Hebel were loaded axially using uniformly spaced external post-tensioning rods, whose axial tension was monitored and was kept constant. Additional external threaded rods, with initial pre-tension less than 0.5 kip (2 kN), were used as external reinforcement. As the Hebel specimens displaced laterally in-plane, they rotated about the compression toe, and tensile forces increased in the rods on the tension side. The rods on the compression side of the wall were not initially post-tensioned, so their force did not decrease as the force in the tension rods increased. Increasing the force in the tension rods without decreasing the force in the compression rods was equivalent to applying an additional compressive axial load to the wall. The axial load used to evaluate the behavior of the Hebel specimens at each state was the initial axial load plus the summation of tensile forces in the rods at that state.

Key Behavioral Modes of AAC Shear Wall Specimens

An AAC shear wall can exhibit the following behavior modes: flexural cracking; web-shear cracking; flexure-shear cracking; nominal flexural capacity; crushing of the diagonal strut; and sliding shear. Depending on the level of axial load and prescriptive reinforcement, these modes can occur singly or in combination. In the following sections, the prediction of each mode is discussed.

Flexural Cracking of AAC Shear Walls -- Flexural cracking was observed in 11 shear wall specimens tested at UT Austin. 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, these flexural cracks formed between the AAC and the mortar leveling bed, indicating tensile bond failure between the two materials. The flexural cracks occurred in both ends of the walls, because the walls were subject to reversed cyclic load. The first occurrence of a flexural crack is recorded as the first tested \( V_{cr} \), while the second occurrence of a flexural crack is recorded as the second tested \( V_{cr} \). The lateral loads at which flexural cracking was observed and back-calculated tensile bond strength for each shear wall are listed in Table 3. In that table, data for shear wall specimens where shrinkage cracks formed along the bedding mortar joint prior to testing are not presented.

Shear Wall Specimens 4 (first occurrence) and 11 (both occurrences) show the highest modulus of rupture. In both specimens, at least one flexural crack was not observed until it had propagated more than one-quarter the plan length of the wall. Those specimens were not included in the calculation of mean and coefficient of variation. The mean modulus of rupture is 68 psi, and the corresponding 20% fractile is 49.4 psi. Based on the latter value it is proposed that the design value for modulus of rupture not exceed 50 psi if a leveling bed joint is present in the AAC element.

Web-shear Cracking of AAC Shear Walls -- Web-shear cracking is characterized by the formation of an inclined crack in the web of the wall, when the principal tensile stress in the web exceeds the diagonal tensile strength of the AAC. That principal stress is given by Equation (1), in which the normal stress in the wall is \( n \) and the maximum shear stress in the center of the web is \( v \). Substituting the equations for shear stress and axial
stress into the above equation, and solving for the shear, the corresponding web-shear capacity is given by Equation (2):

\[
f_t = \sqrt{\left(\frac{n}{2}\right)^2 + \left(v^2\right)^2} - \frac{n}{2} \quad \text{where} \quad v = \frac{3V}{2l_w t} \quad \text{and} \quad n = \frac{P}{l_w t}
\]

Equation (1)

\[
V_{AAC} = \frac{2l_w t}{3} f_t \left[1 + \left(\frac{P}{f_t l_w t}\right)\right]^{0.5}
\]

Equation (2)

Web-shear cracking was observed in all AAC shear-wall specimens tested at The University of Texas at Austin except Shear Wall Specimen 2 (constructed of vertical panels). In addition, the tests performed by Hebel\(^3\) provide corroborating data on web-shear cracking capacity. The shear strength of the AAC shear-wall specimens was initially predicted using Equation (2). Data on partially mortared specimens are omitted here for brevity but is given in Reference (2).

The ratio of observed to predicted web-shear cracking capacity for Shear Wall Specimen 1 is significantly greater than for the other specimens, and can be considered anomalous. For the remaining specimens, Equation (2) is unconservative.

For the shear walls with fully mortared head joints, the ratios of observed to predicted values of \(V_{AAC}\) range from 0.54 to 1.29, with a mean of 0.69 and a COV of 17% (Figure 7). In that figure, the mean ratio of observed capacity to the predicted capacity is represented by a solid horizontal line. The normal distribution with the same mean and COV as the test data is also plotted on Figure 7. The lower 10% fractile of that distribution, shown by a dashed horizontal line, is 0.54.

Equation (2) was multiplied by 0.54 so that it would correspond to the lower 10% fractile of the ratios of observed to predicted capacities. In addition, the following substitution, \(f_t = 2.4\sqrt{f_{AAC}}\), was incorporated to produce Equation (3) (Reference 5). A similar derivation exists for the case of shear walls with unmortared head joints; the result is presented in Equation (4) (Reference 5). Using these equations the observed web-shear cracking capacity is slightly greater than the predicted web-shear cracking capacity for walls with fully mortared head joints, and with unmortared head joints. In this section the following notation is used: \(f_{AAC}\)=compressive strength of AAC; \(l_w\)=wall length in plan; \(P_u\)=axial load in wall; \(t\)=nominal thickness of wall; and \(V_{AAC}\)=lateral load capacity of wall as governed by AAC.

\[
V_{AAC} = 0.86 \ell_w t \sqrt{f_{AAC}} \left[1 + \frac{P_u}{2.4\sqrt{f_{AAC}} \ell_w t}\right]^{0.5}
\]

Equation (3)
Flexure-Shear Cracking of AAC Shear Walls -- A flexure-shear crack begins as a horizontal crack at a height of about one-half the plan length of the wall ($l_w$) above the base of the wall, and then propagates diagonally through the center of the wall. The formation of this crack is governed by the flexural tensile stress in the wall (Equation (5)).

$$\sigma = \frac{M}{S_x} - \frac{P}{A_n}$$

Equation (5)

Based on experiments with reinforced concrete shear walls, the controlling horizontal crack develops at a height of about $l_w/2$. Therefore, the moment at the crack, $M_{flcr}$ is

$$M_{flcr} = M - \frac{VL}{2}$$

where $M$ is the moment at the base. Equation (6) presents the base shear at the formation of the flexural portion of the flexure-shear crack. In Equation 5 and 6 the following additional notation is used: $A_n$=area of the wall in plan; $M$=design moment in wall; $f_r$=modulus of rupture of AAC; $S_x$=section modulus of a shear wall; $V$=design shear in wall; $V_{flcr}$=capacity of wall as governed by flexural cracking; and $t$=maximum tensile stress in wall.

$$V_{flcr} = \frac{S_x \left( f_r + \frac{P}{l_w t} \right)}{\frac{M}{V} - \frac{l_w}{2}}$$

Equation (6)

ACI 318-02 uses a conservative (low) flexural tensile strength of $6 \sqrt{f'_c}$ (US customary units) substituted into Equation (6); experiments have shown an additional force of $0.6 \sqrt{f'_c} \cdot t$ is required to develop the crack.

Flexure-shear cracking was observed in the 6 flexure-dominated shear wall specimens. The flexural portion of the flexure-shear crack always formed first in the horizontal joint. Based on the location of the flexural crack, the load can be predicted by Equation (6). For AAC the modulus of rupture was calculated using $f_r = 2f_t$ and the tested splitting tensile strength. This value was used in Equation (6). For flexure-dominated shear wall specimens, with the exception of Shear Wall Specimen 14a, the ratio of observed versus predicted capacity ranges from 0.6 to 1.3. The mean ratio is 0.86 with a COV of 36%.
The observed capacity is lower than that predicted because the failure occurred in the joint between the AAC and the thin-bed mortar rather than in the AAC material itself. A relationship for the tensile bond strength $f_{bond}$ between AAC and thin-bed mortar was determined based on tests performed at UAB (Reference 5). Equation (7) presents the tensile bond strength for compressive strengths greater than 450 psi (3.1 MPa). This indicates that for mid- to high-strength AAC the tensile bond strength, $f_{bond}$, is lower than the modulus of rupture, $f_r$. The results of using the tensile bond strength rather than the modulus of rupture in Equation (6) for the remaining specimens are presented in Table 4. With the exception of Shear Wall Specimen 14a, the resulting average ratio of observed $V_{fcr}$ to predicted $V_{fcr}$ is 1.1 with a COV of 19%.

Shear Wall Specimen 14a exhibited flexural cracks at the west side of the base of the wall prior to testing. These cracks are presumed to have occurred while moving the top of the wall (out-of-plane) approximately 1 in. (25 mm) to the east to align the rams and loading beam.

$$f_{bond} = 0.04 \cdot f_{AAC} + 66$$

Equation (7)

Flexural cracking did not decrease the strength or stiffness of the specimens. In each case at least one load cycle was completed before a significant loss of stiffness was observed. Furthermore, the vertical reinforcement was sufficient to carry the load after flexure-shear cracking occurred. Based on these conclusions, no limiting design equations are proposed for flexure-shear cracking.

**Nominal Flexural Capacity of AAC Shear Walls** -- Observed versus predicted nominal flexural capacities can be compared for flexure-dominated Shear Wall Specimen 14a, 14b, 15a and 15b. During the test of Shear Wall Specimen 13 and Shear Wall Specimen 16, the actuators used to apply the constant axial load inadvertently reached the end of their travel. As increasing lateral drifts were applied, axial load on the wall inadvertently increased. To successfully interpret those test results, the probable axial load applied to the walls was back-calculated from the predicted flexural capacity, removing those two tests from consideration for verifying observed versus predicted flexural capacity.

The nominal flexural capacity was calculated using a steel yield strength of 75 ksi (490 MPa), from mill reports. Traditional flexural theory was used with a maximum useful strain of 0.003 in the AAC [5], and a value for $\beta_1$ of 0.67 [6]. The ratios of observed to predicted strength range from 1.11 to 1.29, with an average of 1.19 and a COV of 5.8%. The results may have been consistently increased due to strain hardening in the steel.

**Crushing of the Diagonal Strut of AAC Shear Walls** -- In addition to being idealized structurally as a beam-column or as a diagonal tension element, an AAC shear wall can also be idealized using a strut-and-tie mechanism, in which load is transferred to the foundation through a compressive diagonal strut (Figure 8). The compressive force in the diagonal strut is equilibrated at the base of the wall by the frictional resistance and
vertical component of compression in the diagonal strut. The diagonal strut crushes when the compressive stress exceeds the compressive strength of the AAC.

A free-body diagram of the compressive strut is shown in Figure 9. The free-body diagram of Figure 9a) shows the forces at the ends of the strut; the free-body diagram of Figure 9b) shows the force inside the strut. The force in the strut is a function of the geometry of the wall and horizontal projection of the diagonal strut. The geometry of a shear wall specimen with aspect ratio of 0.6 and horizontal projection of the strut \(l_{strut} \) equal to one-quarter of the plan length of the wall, is shown in Figure 9c). From geometry the force in the strut will be 1.7 times the vertical reaction (ratio of diagonal leg to vertical leg of equivalent triangle). For a squat wall, the diagonal strut can crush at lateral loads smaller than those corresponding to the nominal flexural capacity. Because of the inclination of the strut, the force in the compression diagonal of a squat wall can be much higher than the flexural compression in the wall toe.

Crushing of the diagonal strut was observed in Shear Wall Specimen 1. The length of crushing extended one quarter of the plan length of the wall. Based on the applied load at crushing and the geometry of the wall, Equation (8) was calibrated. In Equation 8, two new variables are introduced: \(h\)=height of wall and \(w_{strut}\)=width the compressive strut.

\[
V_{AAC} = 0.9 f_{AAC} tw_{strut} \frac{h\left(\frac{3}{4} l_{w}\right)}{h^2 + \left(\frac{3}{4} l_{w}\right)^2}
\]

Equation (8)

In Shear Wall Specimens 3, 4, 5, 6, 7 and 9 crushing of the diagonal strut was avoided by limiting the axial load. Since the model was adequate for walls with aspect ratios less than 1.5, that aspect ratio is used as an upper limit to the proposed Code equation.

**Sliding Shear Capacity of AAC Shear Walls** -- An AAC shear wall constructed of horizontal panels or masonry-type units exhibits a bed-joint crack when the shear stress on the bed joints exceeds the interface shear capacity, \(\nu\). After the crack forms the shear is resisted by the vertical reinforcement and by the frictional forces due to the axial load (Figure 10).

In the traditional shear friction mechanism, sliding over a rough interface causes the crack at the interface to widen, stressing any reinforcement crossing the interface and providing additional clamping force. Under reverse cyclic loading of AAC, the roughness of the bed joints can decrease, as a result of which resistance to sliding shear is provided primarily by dowel action of reinforcement crossing the bed joints. Sliding was observed in tests of Shear Wall Specimen 4 and the Two-story Assemblage Specimen. In both cases the vertical reinforcement contributed significantly to the capacity for several cycles until local crushing and spalling of the grout in the 3 in. (76 mm) diameter cells and surrounding AAC began and continued throughout the test, reducing the effectiveness of the reinforcement. Based on the observed tests results, the design
recommendation is to neglect the contribution of dowels and longitudinal steel to sliding shear resistance. The proposed design equation for sliding shear is presented in Equation (9).

\[ V_{ss} = \mu P_u \]

Equation (9)

Conclusions from Part 2

Based on the results of experimental testing of AAC shear wall specimens, reliable design models and predictive equations have been developed and calibrated to describe the behavior of AAC shear walls under monotonic and reversed cyclic shear loads. The results show agreement with low COV’s for shear walls with a wide range of aspect ratios and axial loads. This research allows appropriate design provisions to be developed for AAC shear walls in general.

OVERALL SUMMARY AND CONCLUSIONS

A first part of a consistent technical basis for the design of autoclaved aerated concrete (AAC) structures is presented. The basis includes an overview of an overall strategy for development of design provisions in the context of the US code framework, and the development and testing of shear wall specimens at The University of Texas at Austin.

Acknowledgments

Most of the information reported here is based on a research project on the Seismic Behavior of Autoclaved Aerated Concrete, conducted at The University of Texas at Austin, under the sponsorship of the Autoclaved Aerated Concrete Products Association.

References


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9) *Specifications for Masonry Structures (ACI 530.1-02 / ASCE 6-02 / TMS 602-02)*, American Concrete Institute, Farmington Hills, Michigan, American Society of Civil Engineers, Reston, Virginia, and The Masonry Society, Boulder, Colorado, 2002.

10) ACI 318, Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, 2002.

**Table 1 Typical mechanical and thermal characteristics of AAC**

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>AAC</th>
<th>Conventional Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>density,pcf (kg/m³)</td>
<td>25 - 50 (400 - 800)</td>
<td>80 - 150 (1280 - 2400)</td>
</tr>
<tr>
<td>compressive strength, f_c, psi (MPa)</td>
<td>360 - 1090 (2.5 - 7.5)</td>
<td>1000 - 10000 (6.9 - 69)</td>
</tr>
<tr>
<td>thermal conductivity, Btu-in/ft²-hr-F</td>
<td>0.75 - 1.20</td>
<td>6.0 - 10</td>
</tr>
<tr>
<td>fire rating, hours</td>
<td>≤ 8</td>
<td>≤ 6</td>
</tr>
</tbody>
</table>
## Table 2: Details of shear wall specimens tested at UT Austin

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Failure Mode</th>
<th>AAC Material</th>
<th>Supplier</th>
<th>Length (in. (m))</th>
<th>Height (in. (m))</th>
<th>Thickness (in. (m))</th>
<th>Aspect Ratio</th>
<th>Int. Vertical Reinf.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Shear</td>
<td>Horiz. Panels</td>
<td>Contec 1</td>
<td>240 (6.1)</td>
<td>154 (3.9)</td>
<td>8 (0.2)</td>
<td>0.64</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>Shear</td>
<td>Vert. Panels</td>
<td>Ytong 1</td>
<td>240 (6.1)</td>
<td>154 (3.9)</td>
<td>8 (0.2)</td>
<td>0.64</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>Shear</td>
<td>Blocks</td>
<td>Ytong 2</td>
<td>240 (6.1)</td>
<td>151 (3.8)</td>
<td>8 (0.2)</td>
<td>0.63</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>Shear</td>
<td>Horiz. Panels</td>
<td>Matrix 1</td>
<td>240 (6.1)</td>
<td>154 (3.9)</td>
<td>8 (0.2)</td>
<td>0.64</td>
<td>#5 (16 mm) / 48 in. (1.2 m)</td>
</tr>
<tr>
<td>5</td>
<td>Shear</td>
<td>Blocks</td>
<td>Contec 2</td>
<td>240 (6.1)</td>
<td>154 (3.9)</td>
<td>8 (0.2)</td>
<td>0.63</td>
<td>No</td>
</tr>
<tr>
<td>7</td>
<td>Shear</td>
<td>Blocks</td>
<td>Ytong 2</td>
<td>144 (3.6)</td>
<td>151 (3.8)</td>
<td>8 (0.2)</td>
<td>1.05</td>
<td>No</td>
</tr>
<tr>
<td>8</td>
<td>Shear</td>
<td>Horiz. Panels</td>
<td>Matrix 2</td>
<td>96 (2.4)</td>
<td>154 (3.9)</td>
<td>8 (0.2)</td>
<td>1.60</td>
<td>No</td>
</tr>
<tr>
<td>9</td>
<td>Shear</td>
<td>Blocks</td>
<td>Matrix 1</td>
<td>240 (6.1)</td>
<td>151 (3.8)</td>
<td>8 (0.2)</td>
<td>1.63</td>
<td>No</td>
</tr>
<tr>
<td>11</td>
<td>Shear</td>
<td>Blocks</td>
<td>Ytong 2</td>
<td>48 (1.2)</td>
<td>151 (3.8)</td>
<td>8 (0.2)</td>
<td>3.15</td>
<td>No</td>
</tr>
<tr>
<td>13</td>
<td>Flexure</td>
<td>Horizontal Panels</td>
<td>Ytong 1</td>
<td>72 (1.8)</td>
<td>154 (3.9)</td>
<td>8 (0.2)</td>
<td>2.13</td>
<td>#5 (16 mm) / 12 in. (0.6 m) from ends</td>
</tr>
<tr>
<td>14a</td>
<td>Flexure</td>
<td>Horizontal Panels</td>
<td>Babb 1</td>
<td>56 (1.4)</td>
<td>154 (3.9)</td>
<td>10 (0.3)</td>
<td>3.2</td>
<td>#5 (16 mm) / 4 in. (1.0 m) from ends</td>
</tr>
<tr>
<td>14b</td>
<td>Flexure</td>
<td>Horizontal Panels</td>
<td>Babb 1</td>
<td>56 (1.4)</td>
<td>154 (3.9)</td>
<td>10 (0.3)</td>
<td>3.2</td>
<td>#5 (16 mm) / 4 in. (1.0 m) from ends</td>
</tr>
<tr>
<td>15a</td>
<td>Flexure</td>
<td>Vertical Panels with End Blocks</td>
<td>Babb 1</td>
<td>112 (2.8)</td>
<td>154 (3.9)</td>
<td>10 (0.3)</td>
<td>1.4</td>
<td>#5 (16 mm) / 8 in. (2.0 m) from ends</td>
</tr>
<tr>
<td>15b</td>
<td>Flexure</td>
<td>Vertical Panels with End Blocks</td>
<td>Babb 1</td>
<td>112 (2.8)</td>
<td>154 (3.9)</td>
<td>10 (0.3)</td>
<td>1.4</td>
<td>#5 (16 mm) / 8 in. (2.0 m) from ends</td>
</tr>
<tr>
<td>16</td>
<td>Flexure</td>
<td>Vertical Panels with U End Blocks</td>
<td>Babb 1</td>
<td>112 (2.8)</td>
<td>154 (3.9)</td>
<td>10 (0.3)</td>
<td>1.4</td>
<td>#5 (16 mm) / 8 in. (2.0 m) from ends</td>
</tr>
<tr>
<td>Assemblage</td>
<td>Flexure</td>
<td>Vertical Panels with U End Blocks</td>
<td>Babb 2</td>
<td>112 (2.8)</td>
<td>154 (3.9)</td>
<td>10 (0.3)</td>
<td>1.4</td>
<td>#5 (16 mm) / 8 in. (2.0 m) from ends</td>
</tr>
</tbody>
</table>

## Table 3: Calculated modulus of rupture of AAC shear walls tested at UT Austin

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

Mean: 68.3  
COV (%): 33
Table 4: Results in flexure-shear cracking of AAC flexure-dominated shear wall specimens using tensile bond strength of material

<table>
<thead>
<tr>
<th>Specimen</th>
<th>P kips (kN)</th>
<th>Tested $V_{ter}$ North kips (kN)</th>
<th>Predicted $V_{ter}$ North kips (kN)</th>
<th>Tested $V_{ter}$ South kips (kN)</th>
<th>Predicted $V_{ter}$ South kips (kN)</th>
<th>Observed $V_{frx}$/Predicted $V_{ter}$ North kips (kN)</th>
<th>Observed $V_{frx}$/Predicted $V_{ter}$ South kips (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>25 (111)</td>
<td>9.6 (43)</td>
<td>6.9 (31)</td>
<td>10.1 (45)</td>
<td>6.8 (30)</td>
<td>1.39</td>
<td>1.48</td>
</tr>
<tr>
<td>14a</td>
<td>5 (22)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>2.5 (12)</td>
<td>2.5 (12)</td>
</tr>
<tr>
<td>14b</td>
<td>5 (22)</td>
<td>4.9 (22)</td>
<td>4.8 (22)</td>
<td>4.6 (20)</td>
<td>4.8 (22)</td>
<td>1.01</td>
<td>0.95</td>
</tr>
<tr>
<td>15a</td>
<td>25 (111)</td>
<td>21.5 (96)</td>
<td>21.5 (96)</td>
<td>24 (107)</td>
<td>26.4 (118)</td>
<td>1.00</td>
<td>0.91</td>
</tr>
<tr>
<td>15b</td>
<td>25 (111)</td>
<td>20 (89)</td>
<td>20.6 (92)</td>
<td>17.5 (78)</td>
<td>19.7 (88)</td>
<td>0.97</td>
<td>0.89</td>
</tr>
<tr>
<td>16</td>
<td>25 (111)</td>
<td>24 (107)</td>
<td>21.5 (96)</td>
<td>21.8 (97)</td>
<td>21.9 (97)</td>
<td>1.15</td>
<td>1.00</td>
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</table>

Mean 1.07

COV (%) 19

Table 5: Observed versus predicted nominal shear capacities based on nominal flexural capacity

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Predicted $V_{Ma}$ kips (kN)</th>
<th>Observed $V_{Ma}$ South kips (kN)</th>
<th>Observed $V_{Ma}$ North kips (kN)</th>
<th>Observed / Predicted $V_{Ma}$ South</th>
<th>Observed / Predicted $V_{Ma}$ North</th>
</tr>
</thead>
<tbody>
<tr>
<td>14a</td>
<td>8.5 (38)</td>
<td>9.4 (42)</td>
<td>NA</td>
<td>1.11</td>
<td>NA</td>
</tr>
<tr>
<td>14b</td>
<td>8.5 (38)</td>
<td>9.9 (44)</td>
<td>10.1 (45)</td>
<td>1.16</td>
<td>1.19</td>
</tr>
<tr>
<td>15a</td>
<td>23.9 (106)</td>
<td>28.8 (128)</td>
<td>30.1 (134)</td>
<td>1.21</td>
<td>1.26</td>
</tr>
<tr>
<td>15b</td>
<td>23.9 (106)</td>
<td>26.7 (119)</td>
<td>30.9 (137)</td>
<td>1.12</td>
<td>1.29</td>
</tr>
</tbody>
</table>

Average 1.19

COV (%) 5.8

Figure 1: Cellular structure of AAC
Figure 2: Sample AAC elements

Figure 3: Leveling bed of ASTM C270 mortar, and shims as needed
Figure 4: Laying AAC masonry units using thin-bed mortar

Figure 5: Overall framework for national design basis for AAC structures

Figure 6: Test setup for shear wall specimens (UT Austin)
Figure 7: Ratios of observed to predicted (Equation (2)) web-shear cracking capacities for AAC shear-wall specimens with fully mortared head joints.

Figure 8: Diagonal compressive strut in an AAC shear wall.

Figure 9: Relationship of forces in the diagonal strut.
Figure 10: Sliding shear mechanism in an AAC shear wall with horizontal panels
Technical Justification for Proposed Design Provisions for AAC Structures: Assemblage Test and Development of R and C_d Factors

by R. E. Klingner, J. E. Tanner, and J. L. Varela

Synopsis: This paper summarizes the final phases of the technical justification for proposed design provisions for AAC structures in the US. It is divided into two parts. The first part describes the design and testing of a two-story, full-scale AAC shear wall specimen that was designed and tested at The University of Texas at Austin, under reversed quasi-static loads representative of those experienced in a strong earthquake. The specimen withstood repeated reversed cycles to story drifts of about 0.3%, and displacement ductility ratios of about 3. The specimen conformed with the two main objectives. Those objectives were: 1) to show that the behavioral models developed for the shear walls also govern in a building; and 2) to demonstrate that a squat wall can exhibit failure governed by flexure.

The second part describes the development of R and C_d factors for seismic design of AAC structures. The seismic force-reduction factor (R) specified in seismic design codes is intended to account for energy dissipation through inelastic deformation (ductility) and structural over-strength. The factor (R) is based on observation of the performance of different structural systems in previous strong earthquakes, on technical justification, and on tradition. For structures of autoclaved aerated concrete (AAC), the force-reduction factor (R) and the corresponding displacement-amplification factor (C_d) must be based on laboratory test results and numerical simulation of the response of AAC structures subjected to earthquake ground motions. The proposed factors must then be verified against the observed response of AAC structures in strong earthquakes. The objectives of this paper are: (1) to present a general procedure for selecting values of the factors (R) and (C_d) for use in the seismic design of structures; and (2) using that procedure, to propose preliminary values of the factors (R) and (C_d) for the seismic design of AAC shear-wall structures. The general procedure is based on comparing the predicted ductility and drift demands in AAC structures, as functions of the factors (R) and (C_d), with the ductility and drift capacities of AAC shear walls, as observed in quasi-static testing under reversed cyclic loads. Nonlinear numerical simulations are carried out using hysteretic load-displacement behavior based on test results, and using suites of natural and synthetic ground motions from different seismically active regions of the United States.

Keywords: autoclaved aerated concrete; cellular concrete; design; earthquake
PART 1: DESIGN AND TESTING OF TWO-STORY AAC ASSEMBLAGE SPECIMEN

Introduction

The Two-story Assemblage Specimen was tested on August 12, 2002. The specimen consisted of two flanged walls connected by floor slabs (Figure 1). The walls were constructed of vertically oriented autoclaved aerated concrete (AAC) panels with internal reinforcement and additional field-placed longitudinal reinforcement, and the floor slabs were constructed of internally reinforced AAC panels. On the upper level, the floor panels were oriented longitudinally; on the lower level, transversely. A complete description of the specimen is provided in Reference 1.

Reinforcing Details

The reinforcement in the shear walls of the Two-story Assemblage Specimen consisted of flexural (longitudinal) reinforcement and foundation dowels (Figure 2). The flexural reinforcement continued up the height of the specimen with a splice just above the first-story slab (first elevated slab). The dowels extended 24 in. (61 mm) above the foundation, and were included to increase the sliding shear capacity of the specimen. They were also placed at the level of the first elevated slab to prevent sliding at the bed of leveling mortar placed between the vertical panels and the slab.

Instrumentation and Data Acquisition

Instrumentation was used to measure global and local behavior of the Two-story Assemblage Specimen. Global behavior is characterized by the in-plane force-displacement relationship. The horizontal displacement of each wall was measured at each story using string potentiometers. The applied load was measured through pin load cells which had been successfully used in previous tests of shear wall specimens at UT Austin. Applied loads were verified using pressure transducers.
The local behavior of individual components was measured and will be used to characterize the deformation patterns in the specimen. The following measurements were recorded for both stories of the east and west shear walls:

- Vertical deformations of each wall;
- Diagonal deformation of each story in each wall;
- Profile of floor-slab displacements with respect to the loading beam; and
- Slip between loading equipment and specimen.

The vertical displacements were measured to assess flexural deformations. The diagonal displacements were used to calculate the shearing deformation in each wall. Profiles of floor-slab displacements were measured to determine abnormal deformation patterns or indicate slip between floor panels on the second level slab. Slip between elements was measured using linear potentiometers mounted between the adjacent elements. Slip values were used to detect relative movement between the loading beams and the floor slabs, between the walls and the foundation, and between the floor slabs and the walls. Slip at other interfaces was determined visually using relative movement of the ends of lines drawn across the interfaces.

Data were acquired in real time using a Hewlett Packard 3852 scanner. Each data recording was assigned a load point number in ascending order. As damage occurred in the specimen, the marks were labeled with a load point to determine the sequence of damage in the wall.

Material Tests

Tests were performed to predict the splitting tensile strength and the compressive strength of the AAC. ASTM C1006 tests performed on AAC modular blocks from the same shipment of AAC indicated a splitting tensile strength of 45 psi (0.31 MPa) with a COV of 15% (moisture content of 16%). Compression tests performed on 4 in. by 8 in. cylinders cored from the same units and air-dried to a moisture content of 6% indicated an average strength is 495 psi (3.4 MPa) with a COV of 6%. A yield strength of 75 ksi (517 MPa) was used; this is consistent with the mill reports for the longitudinal reinforcement.

Summary of Loading Program

Lateral load was applied parallel to the webs of the shear walls, through reinforced concrete loading beams attached to the slabs. Equal load was applied at each story based on several nonlinear analyses performed using the nonlinear analysis program CANNY99 (Reference 2). The lateral loading history consisted of a series of reversed cycles to monotonically increasing maximum load or displacement. The actuators were loaded manually to target load values, based on predicted loads of events that would cause a significant change in the behavior of the specimen. In this case the predicted major events were flexural cracking and the nominal flexural capacity. The loading history used for each actuator is shown in Figure 3. After the maximum load was reached, the
specimen was loaded based on target displacements for the tip of the wall. These were 0.5 in. (13 mm), 0.8 in. (20 mm) and 1.5 in. (38 mm).

A constant axial load was applied through the self-weight of the specimen and the loading equipment. The total axial load on the specimen was 60 kips (267 kN). That load level was intended to represent the axial load on the walls of an approximately 4-story AAC building.

Summary of Specimen Behavior

Overall specimen behavior is summarized in Figure 4. Total base shear is the summation of the equal shears applied to each floor level. Positive displacements are to the south; negative, to the north. The load-displacement relationships for the east and west walls are shown separately in Figure 5.

The following behaviors were observed in the Two-story AAC Assemblage:

- flexural cracking at the base;
- minor vertical cracking on the north end of the east wall;
- web-shear cracking in the webs of both walls in the lower story;
- yielding of the flexural reinforcement at all four corners of the assemblage;
- separation of the flanges from the webs in the lower story; and
- separation of the vertical joint at the location of reinforcement at the northeast corner.

Flexural cracking was observed at the base of the west wall at a total base shear of 48.4 kips (215 kN), corresponding to a calculated tensile bond stress of 31.5 psi (0.2 MPa). Flexural cracking was observed at the base of the east wall at a total base shear of 81.6 kips (364 kN), corresponding to a calculated tensile bond stress of 62.7 psi (0.4 MPa). Both values fall below the average value of the 14 shear wall specimens previously tested at UT Austin, which was 73 psi (0.50 MPa) with a COV of 29% (Reference 1). This could be attributed to shrinkage cracking at the leveling bed, or to a reduced area of leveling bed mortar. If this additional bond-strength data from the Two-story Assemblage Specimen were combined with data from the shear walls, the average tensile bond strength would decrease to 68 psi (0.47 MPa) and the COV would increase to 33%.

At the location of flexural reinforcement at the north end of the east wall, a vertical crack divided the wall into two individual walls. If the in-plane lateral stiffness of the specimen were dominated by flexure, this crack would significantly decrease the stiffness and strength of the specimen. For the specimen, the ratio of shear stiffness to flexural stiffness is 1.35. Since this value is close to one, the shearing and flexural deformations contribute about equally to the flexibility of the specimen. Because the in-plane lateral stiffness of the specimen was influenced about equally by shear and by flexure, the stiffness did not change much as a result of the vertical crack. Also, since the crack occurred two feet from the flange, the loss of flexural stiffness was less than it would have been had the crack occurred near the middle of the web.
Web-shear cracking was first observed in the east wall while loading to the north at a total base shear of 104.2 kips (464 kN), and while loading to the south, at a total base shear of 117 kips (520 kN). The corresponding values for the west wall are 119 kips (530 kN) and 117 kips (520 kN). Tests conducted to confirm the splitting tensile strength gave an average value of 45 psi (0.3 MPa) with a COV of 15%. This corresponds to a total base shear capacity of 118.8 kips (528 MPa) at a total axial load of 60 kips (267 kN). This predicted value corresponds to mean capacity expressed in Equation (1), as previously proposed based on shear wall tests at UT Austin (Reference 1). The following notation is used in Equation 1: $l_w =$ plan length of the wall; $t =$ thickness of the shear wall; $f_t =$ splitting tensile strength, $P_u =$ factored design axial force in shear wall.

$$V_{AAC} = 0.4 \frac{l_w}{t} f_t \sqrt{1 + \frac{P_u}{f_t l_w t}}$$

As the second set of web-shear cracks formed, the flexural reinforcement began to yield. Based on strain gage readings, the flexural reinforcement in the east wall yielded at a total base shear of 118 kips (525 kN) while loading to the north and to the south. The flexural reinforcement in the west wall yielded at a base shear of about 130 kips (580 kN) in each direction.

The initially predicted total base shear at flexural yielding at the base of each wall, excluding the effect of dowels, under the applied axial load of 60 kips (267 kN), was 90 kips (344 kN). The base shear is determined by dividing the flexural capacity by the effective height or M/V ratio. This method is illustrated in Figure 6. The increase in observed capacity is due to the dowels. The base shear at yielding can be predicted based on the flexural capacity at two critical sections:

- at the base, considering the contribution of the dowels to the flexural capacity; and
- at a critical section just above the ends of the dowels.

Including the contribution of the dowels would significantly increase the flexural capacity at the base. Assuming the critical section at the base, the base shear capacity at yield would be determined by calculating the flexural capacity at yield and converting it to an equivalent base shear yield capacity by dividing by the M/V ratio presented in Figure 6. Assuming the critical section at the point where the dowels end, the flexural capacity would be calculated without the dowels, and would be converted to a base shear yield capacity by dividing by the reduced lever arms between each load and the critical section. The results of these calculations are presented in Table 1. The base shear at yielding of the flexural reinforcement falls between the limiting cases defined by the two critical sections noted above.

After the flexural reinforcement yielded, both walls exhibited flexural behavior, consisting largely of rigid-body rocking. Vertical displacements were observed at the wall bases on the tension side, due to yielding and bond deterioration of the tensile
reinforcement. Crushing of the compression toe was avoided, due to lateral support by the flanges. The maximum loads observed in each wall was 136 kips (605 kN) loading to the south and 144 kips (641 kN) while loading to the north. The tested flexural capacity of each wall falls between the bounds presented in Table 1. After flexural yielding, distributed web-shear cracks continued to form in the walls at the lower level.

Diagonal cracks formed around the dowels, separating those dowels from the webs of the AAC shear walls (Figure 7). This reduced the effectiveness of the dowel action, which in turn reduced the sliding-shear capacity of each wall to 48 kips (214 kN) at an axial load of 30 kips (134 kN). Degradation of dowel action is also identified by spalling of AAC around the diagonal cracks (Figure 7). Slip between the AAC shear walls and the foundation exceeded 0.5 in. (13 mm) after LP 677, corresponding to a total base shear of 142.6 kips (634 kN). The wall displacement, corrected for this slip, is shown in Figure 8. The final cycle loading to the south contains a correction for slip based on the results of a previous displacement and slip.

After 3 cycles of flexural rocking, to displacement drift ratios of 0.32% (loading south) and 0.24% (loading north), vertical cracks began to form at the interface between the web and the flanges. As the displacements increased, the flange panel did not slide with the web in the direction of loading, resulting in local damage to the flange and finally instability of the flange at both the north and south ends of the specimen. At the north end, the flange damage was accompanied by a large vertical crack in the east web (Figure 9). Testing of the Two-story Assemblage Specimen was halted due to this damage. Final cracking patterns for each exterior face of the specimen are shown in Figure 10 through Figure 12. The cracks shown in grey formed at the time of yielding of the flexural reinforcement; subsequent cracks are shown in black.

Summary of Response for Part 1

Including base slip, the Two-story AAC Assemblage Specimen reached drift ratios between 0.7% and 0.85%. Final displacement ductilities (final displacement divided by the displacement at yielding of the flexural reinforcement), ranged from 8.3 to 11.7 (Table 2). After removing the base slip, the Two-story Assemblage Specimen reached drift ratios between 0.24% and 0.42%, and final displacement ductilities ranged from 2.8 to 5.8 (Table 3). For design purposes, these results justify an assumption of an available flexural ductility of at least 3.0, reasonably consistent with that observed in previous tests of flexure-dominated AAC shear walls at UT Austin (Reference 1).

PART 2: DEVELOPMENT OF R AND C₄ FACTORS FOR SEISMIC DESIGN OF AAC STRUCTURES

Introduction

The seismic design philosophy of current United States building codes allows most structures to undergo inelastic deformations in the event of strong earthquake ground motions. As a result, the design lateral strength can be lower than that required to
maintain the structure in the elastic range. In the International Building Code 2000 (IBC 2000, Reference 3) the seismic force-reduction factor (R) and the displacement-amplification factor (Cd) are expressed as a response modification coefficient and a deflection amplification factor, respectively. The values of R and Cd are based on observations of the performance of different structural systems in previous strong earthquakes, on technical justification, and on tradition.

Some research has been completed on the seismic behavior of autoclaved aerated concrete (AAC) walls, primarily focusing on the behavior of walls of AAC masonry-sized units. For example, a research project studied the performance of AAC wallets, and walls under lateral loads (Reference 4); another tested the flexural behavior of non-load bearing AAC walls (Reference 5). Other research addressed the behavior and bearing capacity of AAC walls with confining concrete elements (Reference 6), and the out-of-plane capacity of AAC masonry walls (Reference 7).

Based on the literature review conducted, there is insufficient prior research on the seismic performance of AAC structures to develop seismic design provisions. Sufficient information, however, has been acquired to permit the development of design provisions in areas with low seismic risk, such as Florida and Texas. Because there is insufficient prior research to verify the seismic performance of AAC structures, the selection of the seismic factors (R) and (Cd) for AAC structures needs to be based on laboratory test results and the simulation of the seismic behavior of AAC structures subjected to earthquakes representative of different seismic zones of the United States.

The objectives of this paper are: (1) to present a general procedure for selecting values of the factors (R) and (Cd) for use in the seismic design of structures; and (2) using that procedure, to propose preliminary values of the factors (R) and (Cd) for the seismic design of AAC shear-wall structures. The general procedure is based on comparing the predicted ductility and drift demands in AAC structures as functions of the ductility reduction factor (Rd), with the ductility and drift capacities of AAC shear walls as observed in quasi-static testing under reversed cyclic loads. Nonlinear numerical simulations are carried out using hysteretic load-displacement behavior based on test results, and using suites of natural and synthetic ground motions from different seismically active regions of the United States. The proposed value of the factor (R) is the product of the factor (Rd) and an overstrength factor (Ωoverstrength), and the proposed value of Cd is a function of the proposed value of R and the overstrength factor (Ωoverstrength). Details are presented in Reference 8.

**General Procedure for Selecting the Ductility Reduction Factor (Rd)**

The factor (R) defined in the IBC 2000 code is the product of the ductility reduction factor (Rd) and structural overstrength factor (Ωoverstrength) (NEHRP 2000). A general procedure to select the ductility reduction factor (Rd) is presented in this section. Selection of the overstrength factor (Ωoverstrength) is presented later. The procedure to select Rd is explained for AAC shear-wall structures but it can be used for any structural system.
1) Select the type of AAC structure (for example, cantilever or coupled-wall structures).
2) Select the preliminary plan geometry of the AAC walls (thickness and plan length) based on architectural layout, other restrictions (such as manufacturer’s limitations on different unit sizes), or both.
3) Select the structure’s number of stories based on the architectural design, the maximum compressive strength of the AAC, or both.
4) Select a tributary width based on architectural plan distribution (for example, wall spacing).
5) Calculate the weights of different stories of the structure using the selected tributary width and the density of the materials (for example, density of AAC, reinforced concrete, or both).
6) Obtain the design spectrum according to the IBC 2000 using the intended geographic location of the structure.
7) Analyze the structure using the modal analysis procedure of the IBC 2000.
8) Calculate the structure’s elastic global drift ratio and compare it with the maximum global drift ratio capacity of 1% selected for AAC shear-wall structures. In this step, a value of ductility reduction factor $R_d$ of one is used ($R_d=1$). If the global drift ratio of the structure exceeds 1% then the wall length needs to be increased, its tributary width decreased, or both.
9) Obtain the bending moments and shear forces acting on the walls. In this step, a value of the factor ($R_d$) of one is used ($R_d=1$).
10) Assume a design flexural capacity of walls equal to the bending moments calculated in Step 9.
11) Select a suite of earthquakes representative of the design response spectrum.
12) Select an earthquake from the suite.
13) Select a value of $R_d$ greater than unity. Redesign the structure for a reduced flexural capacity. For example, if $R_d$ is selected as 2, then the required flexural capacity is reduced by a factor of 2.
14) Calculate the global drift ratio and the displacement ductility demands in the structure using a dynamic nonlinear analysis. In this step, the earthquake selected in Step 12 is used with the design flexural capacity from Step 13.
15) If the global drift ratio is less than 1%, then select a larger value of $R_d$, return to Step 14, and repeat. If the global drift ratio is greater than 1%, then go to Step 14 and repeat with a value of $R_d$ smaller than that used previously, but still greater than unity. Iterate until the global drift ratio demand is equal to 1%.
16) If the displacement ductility is less than the maximum displacement ductility capacity of 3.5 selected for AAC shear-wall structures, then select a larger value of $R_d$, return to Step 14, and repeat. If the displacement ductility is greater than 3.5, then go to Step 14 and repeat with a value of $R_d$ smaller than that used previously, but still greater than unity. Iterate until the displacement ductility demand is equal to 3.5.
17) Repeat this procedure for other earthquakes, other suites of earthquakes and other structures to obtain a set of $R_d$ factors.
Selecting and Designing AAC Shear-Wall Structures

Four AAC structures were selected for evaluation under earthquake ground motions from different seismically active regions of the United States. The four structures were selected as AAC shear-wall structures because shear walls are the major structural elements resisting seismic forces. The AAC structures selected were: a three and a five-story cantilever-wall structure, and a three and a five-story coupled-wall structure. Typical wall dimensions of 240 in. (6.1 m) long, 120 in. (3 m) high and 10 in. (0.25 m) thick were used in every story of each structure. The coupled-wall structures consisted of two cantilever walls connected by coupling beams at every story. All coupling beams were 48 in. (1.2 m) long, 40 in. (1 m) wide and 10 in. (0.2 m) thick.

Each AAC structure was analyzed and designed following the steps described in the procedure to select the factor ($R_d$). The structures were modeled as plane structures. A tributary width of 240 in. (6.1 m) was assumed to calculate the weights of each story. Design spectra for the seismic regions studied were calculated using site classes consistent with the suite of earthquakes selected. Elastic analyses were carried out using the program SAP2000, with a reduced stiffness consistent with that used in the nonlinear analyses.

Selecting and Scaling Suites of Earthquakes

Different suites of earthquake ground motions were selected based on areas with high potential for seismic activity. Zones in the central and eastern US were considered, and also zones in the western US. For the central and eastern US, three suites of earthquakes were selected: Charleston, SC; Carbondale, IL; and Memphis, TN. For the western US, two suites of earthquakes were selected: Los Angeles, CA; and Seattle WA. Each suite of earthquakes consists of ten earthquake ground motions.

Selection of Suites of Earthquakes for the Central and Eastern US -- Charleston, SC is a seismic region where earthquake ground motions of engineering interest are scarce. The use of synthetic earthquake ground motions representative of the seismicity of that region is a good alternative. A model created by Frankel et al. (1996) was used to develop synthetic ground motions representative of the B-C soil class interface of Charleston, SC. The suite of earthquakes for Charleston, SC used in this project was taken from that work, and corresponds to a 2% probability of exceedance in 50 years.

New Madrid is also a seismic zone where strong earthquake ground motions are scarce. Projects RR-1 and RR-2 of the Mid-America Earthquake Center (MAE) involved the development of uniform hazard spectra and synthetic ground motions for three major Mid-American cities: Carbondale, IL, Memphis, TN, and St. Louis, MO (Wen 1999). The ground motions used in this project for Carbondale, IL and Memphis, TN were taken from those projects RR-1 and RR-2. The selected suites are representative of the Soil Profile of Carbondale, IL and Memphis, TN, and correspond to a 2% probability of exceedance in 50 years.
Selection of Suites of Earthquakes for the Western US -- The SAC Phase 2 Steel Project provided suites of earthquake ground motions for three United States cities: Boston, MA, Los Angeles, CA and Seattle, WA (Somerville 1997). The suites of earthquakes for Los Angeles, CA and Seattle, WA used in this project were taken from that SAC project. The selected suites are representative of Soil Class D, and correspond to a 2% probability of exceedance in 50 years.

Scaling of Suites of Earthquakes -- The selected five suites of earthquakes were scaled to represent the design seismic forces. Acceleration response spectra were calculated for each entire suite of earthquakes and compared with corresponding design spectra. For Charleston, Carbondale, and Memphis, acceleration response spectra were compared with corresponding IBC 2000 Site Class C design spectra and for Los Angeles and Seattle, with corresponding IBC 2000 Site Class D design spectra. Each entire suite was scaled using a single scaling factor calculated as follows: (1) Calculate the elastic response spectra for the suite of earthquakes. (2) Calculate the mean spectral accelerations of the response spectra for periods of 0.26 seconds and 0.62 seconds. In this step, periods of 0.26 seconds and 0.62 seconds are used because they represent the natural periods of the three-story and five-story AAC shear-wall structures studied. (3) Calculate a scaling factor for each period as the design spectral acceleration divided by the mean spectral acceleration. (4) The final scaling factor is the average of the two scaling factors calculated in Step 3. Two scaling factors, however, were used for the suite of Charleston because of the large difference between the two scaling factors calculated in Step 3.

Selection of the Maximum Global Drift Ratio and Displacement Ductility Capacities for AAC Shear-Wall Structures

The procedure proposed in this paper to select the factors \( R \) and \( C_d \) is based on a maximum global drift ratio and displacement ductility capacities. The maximum global drift ratio is considered to limit damage and differential movement in AAC shear-wall structures. The maximum displacement ductility is considered to control the amount of inelastic deformation in AAC shear-wall structures. The main objective on selecting drift and ductility capacities is to provide reasonable limits to avoid collapse of AAC shear-wall structures. Both drift ratio and displacement ductility capacities for AAC structures are based on test results.

Maximum Drift Ratio Capacity for AAC Structures -- Six AAC shear wall specimens were tested at the Phil M. Ferguson Structural Engineering Laboratory at the University of Texas at Austin. All the specimens were flexure-dominated walls tested under quasi-static reverse cyclic loads. Physical details and axial load applied for each of the flexure-dominated walls are presented in Table 4.

Two maximum drift ratios corresponding to loading the wall in the south and north directions were selected for each specimen. The maximum drift ratios were defined based on the following criteria: (1) a reduction on the flexural capacity of the AAC wall of more than 10% was observed; and (2) a change on the shape of the hysteretic loop was observed from the corresponding previous load cycle, for example, a large reduction in
the energy dissipated. The maximum drift ratio for each wall is shown in Table 5. Only one maximum drift ratio is presented for Shear Wall Specimen 14a because it was tested under monotonic loading.

The observed maximum drift ratio of 0.4% corresponding to Shear Wall Specimen 16 was low compared with the other observed values. Reasons for this are the following: (1) additional axial load was applied inadvertently during the test; (2) cracking between the end vertical panel and the U blocks at the lower north corner of the wall damaged the north compressive toe. A replica of this specimen will be tested using Heli-fix® ties between the vertical panel and the U blocks along the wall height, to improve the behavior of the compressive toes and the overall performance of the wall.

The observed maximum drift ratio of Shear Wall Specimen 14b was smaller in the north direction than in the south direction. This difference can be attributed to the effect of the cyclic loading on the overall response of the wall and to the large increment in the imposed displacement in consecutive cycles during the test.

A value of maximum global drift ratio of 1% was proposed to avoid collapse of AAC shear-wall structures. This value corresponds to the minimum observed maximum drift ratio of Shear Wall Specimens 13, 14a, 14b, 15a, and 15b. The maximum drift ratio of 0.4% for Shear Wall Specimen 16 was not considered because this low value of drift ratio was associated with failure of the joint between the vertical panel and the U blocks which can be eliminated or improved using walls with flanges, Heli-fix® ties, or both.

**Maximum Displacement Ductility Capacity for AAC Structures** -- Two maximum displacement ductilities corresponding to loading the wall in the south and north directions were selected for each flexure-dominated wall. The maximum displacement ductilities were defined based on the same criteria defined for selecting the maximum global drift ratios. The maximum displacement ductility for each wall is shown in Table 5.

A value of maximum displacement ductility capacity of 3.5 was proposed to avoid collapse of AAC structures. This value corresponds to the 10% lower fractile of the maximum displacement ductilities of Shear Wall Specimens 13, 14a, 14b, 15a and 15b. The maximum displacement ductility of 1.67 for Shear Wall Specimen 16 was not considered for the same reasons presented in the selection of the maximum global drift ratio capacity.

**Nonlinear Analysis**

To select factors (R) and (C_d) for AAC structures, the performance of the four AAC shear-wall structures under the selected suites of earthquakes was evaluated using the nonlinear analysis program CANNY 99 (Reference 2).

**Model for Nonlinear Walls** -- Structures in the program CANNY 99 are idealized as rigid nodes connected by line elements and springs. All structural elements are treated as
massless line elements represented by their centroidal axes, with mass concentrated at the nodes or at the center of gravity of floors. The idealized wall element of CANNY 99 considers the wall as a line element located at the wall centerline. The wall element is idealized using two nonlinear flexural springs, two rigid links, one nonlinear shear spring and one axial spring (Figure 13). The nonlinear flexural springs are located at the top and bottom of the wall centerline. Therefore, all of the nonlinearity is concentrated at the wall ends (lumped nonlinearity).

**Hysteretic Model for Nonlinear Behavior of Walls** -- The hysteretic model selected to represent the behavior of the nonlinear flexural and shear springs was the CANNY CA7 model which uses user-input hysteretic parameters to define the loading and unloading branches, degradation of strength and stiffness, and pinching of the hysteretic loops. The behavior of the nonlinear flexural spring is defined by a moment-rotation curve and the nonlinear shear spring by a force-displacement curve. The behavior of the axial spring was defined by the elastic model EL1 of CANNY 99.

Based on the observed behavior of the six flexure-dominated walls, the hysteretic curve of the nonlinear flexural spring was defined as follows: (1) the initial stiffness is defined using the modulus of elasticity of AAC and a reduced moment of inertia equal to 40% of the gross moment of inertia of the wall; (2) the post-yielding stiffness is selected as 1% of the initial stiffness for the three-story structures and 0.5% for the five-story structures; and (3) the degradation of the unloading stiffness is defined using a hysteretic parameter \( \theta \) of 1 (CANNY 99). Strength degradation and pinching are not including because they were not observed up to a global drift ratio of 1% and a displacement ductility of 3.5.

Based on the observed behavior of eight shear-dominated walls tested at Ferguson Structural Engineering laboratory of The University of Texas at Austin as part of this study, the hysteretic curve of the nonlinear shear spring was defined as follows: (1) the initial stiffness is defined using the shear modulus of AAC and a reduced area equal to 40% of the gross area of the wall; (2) the stiffness after shear cracking is selected as 1% of the initial stiffness; (3) the degradation of the unloading stiffness is defined using a hysteretic parameter \( \theta \) of 1; and (4) the degradation of the shear strength is defined using a hysteretic parameter \( \lambda_u \) of 0.45 and \( \lambda_e \) of 0 (Reference 2). Pinching of the hysteretic loops is not included because this phenomenon was not observed in all the shear-dominated walls.

**Proposed Value of R for AAC Structures**

The procedure described above to select the ductility reduction factor (\( R_d \)) was carried out for the four selected structures using the suites of earthquake representative of Charleston, Carbondale, Memphis, Los Angeles and Seattle. In most cases, values of \( R_d \) of 1, 2, 3 and 4 were assumed in the proposed procedure. Linear interpolation was used among those values to calculate critical values of \( R_d \) (values of \( R_d \) that make the global drift ratio and displacement ductility demands equal to the maximum global drift ratio and displacement ductility capacities). A mean value of the factor \( R_d \) was selected for each different structure and suite of earthquakes, as the minimum value between the
average critical values of $R_d$ based on global drift ratio and displacement ductility capacities. In all cases the critical value of $R_d$ based on displacement ductility was smaller than that based on global drift ratio. In few cases during the nonlinear analyses, the global drift ratio demand for a value of $R=R_d$ of 1 was greater than the global drift ratio capacity of 1%. Therefore, for those particular cases, values of $R_d$ based on that global drift ratio were not selected. Table 6 presents the selected mean values of $R_d$ based on displacement ductility for the different structures and suites of earthquakes.

In Table 6, ST-1W-3S and ST-1W-5S are the three and five-story cantilever-wall structures, and ST-2W-3S and ST-2W-5S are the three and five-story coupled-wall structures. The mean values of $R_d$ presented in Table 6, for the three and five-story cantilever-wall structures were smaller than those corresponding to the three and five-story coupled wall structures. The reason was that the maximum inelastic displacement and displacement ductility demands for the cantilever-wall structures are greater than those corresponding to the coupled-wall structures.

A value of $R_d$ of 2 is proposed for AAC shear-wall structures based on the 10% lower fractile value of the mean values of $R_d$, presented in Table 6. The approach adopted here was to select a value of $R_d$ that would result in structural failure (exceedance of drift or ductility capacities) less than 10% of the time under suites of earthquakes representing in average the design spectra.

The structural overstrength factor ($\Omega_{\text{overstrength}}$) is the product of independent overstrength factors defined as follows (References 9, 10): (1) development of sequential plastic hinges in redundant structures; (2) material strengths higher than those specified in design; (3) strength reduction factors; (4) specified sections and reinforcement patterns greater than those required in design; (5) nonstructural elements; and (6) variation of lateral forces.

For AAC shear-wall structures, independent overstrength factors are proposed as follows: (1) For AAC shear-wall structures, plastic hinges at the base of the walls would form at the same time; that is, the redundancy factor would be equal to 1. (2) Assume yield strength of reinforcing bars 10% higher than specified in design. (3) The strength reduction factor for flexural design of walls is equal to 0.9, corresponding to an overstrength factor of 1.1. (4) Assume selected flexural reinforcement 10% greater than that required in design. (5) Ignore participation of nonstructural elements. (6) The minimum design seismic forces specified in the IBC 2000 for the four selected structures were at least 20% greater than those obtained from the elastic modal spectral analysis. Two probable reasons are: (1) the static analysis is a simplification of the modal spectral analysis; and (2) cracked properties of the walls were used in the modal spectral analyses.

The product of the above independent overstrength factors is equal to 1.6. A value of structural overstrength factor ($\Omega_{\text{overstrength}}$) of 1.5 is proposed for AAC shear-wall structures.
The factor \( R \) is the product of the ductility reduction factor \( R_d \) and the overstrength factor \( \Omega_{\text{overstrength}} \). Using the proposed ductility reduction factor \( R_d \) of 2 and the overstrength factor \( \Omega_{\text{overstrength}} \) of 1.5, a value of the seismic force-reduction factor \( R \) of 3 is proposed for AAC shear-wall structures.

**Proposed Value of \( C_d \) for AAC Structures**

The value of the displacement amplification factor \( C_d \) is defined as the maximum nonlinear displacement during an earthquake \( D_{\text{max}} \) divided by the elastic displacement \( D_e \) calculated using reduced seismic design forces (Reference 9). Maximum inelastic displacements \( D_{\text{max}} \) for the four AAC shear-wall structures should be calculated using a value of \( R \) of 3 and a value of \( \Omega_{\text{overstrength}} \) of 1.5 (Figure 14). In that figure, \( V_e \) is the design lateral force associated with a value of \( R \) of 1, \( V_y \) is the lateral force at which significant yield is observed in the structural system, and \( D_e \) and \( D_y \) are the elastic displacements calculated using \( V_e \) and \( V_y \) respectively. \( V_y \) can be assumed approximately equal to \( V_e \) divided by a value of \( R \) of 3, and multiplied by a value of \( \Omega_{\text{overstrength}} \) of 1.5. This final reduced force is equal to that corresponding to a value of \( R \) of 2 and a value of \( \Omega_{\text{overstrength}} \) of 1 represented in Figure 14 by the idealized nonlinear behavior. Therefore, the inelastic displacements calculated using a value of \( R \) of 2 and a value of \( \Omega_{\text{overstrength}} \) of 1 are similar to those corresponding to a value of \( R \) of 3 and a value of \( \Omega_{\text{overstrength}} \) of 1.5.

Using the maximum inelastic displacements calculated using a value of \( R \) of 3 and a value of \( \Omega_{\text{overstrength}} \) of 1.5 (or value of \( R \) of 2 and \( \Omega_{\text{overstrength}} \) of 1), values of the factor \( C_d \) were calculated as the maximum inelastic displacement \( D_{\text{max}} \) divided by the elastic displacement corresponding to a value of \( R \) of 3 \( (D_e / 3) \). Mean \( C_d \) values were calculated for each different structure and suite of earthquakes studied. Those mean values of \( C_d \) are shown in Table 7.

The average, 10% lower fractile, and 10% upper fractile values of mean values of \( C_d \) were equal to 3.49, 2.79, and 4.19, respectively. The proposed value of the factor \( (R) \) of 3 is greater than the 10% lower fractile value of the mean values of \( C_d \) of 2.79. This result is consistent with IBC 2000 values of \( R \) and \( C_d \) for other structural systems, for which, values of \( R \) are greater or equal to \( C_d \) values.

The value of \( R \) of 3 proposed for the seismic design of AAC shear-wall structures was based on a 10% lower fractile value to be conservative in selecting the final design seismic forces. The value of \( C_d \), however, should be based on an upper fractile value to be conservative in the estimation of the maximum inelastic displacements. If the factor \( (C_d) \) is based on the 10% upper fractile value of 4.19, then the value of \( C_d \) would be greater than the proposed value of \( R \) of 3. To be consistent with the IBC 2000, a preliminary value of \( C_d \) of 3 is proposed for the seismic design of AAC structures.
Summary and Conclusions for Part 2

A general procedure for selecting values of the factors $R$ and $C_d$ for use in the seismic design of different structural systems is presented. The proposed procedure is based on comparing the predicted ductility and drift demands in AAC structures as functions of the ductility reduction factor $R_d$, with the ductility and drift capacities of AAC shear walls as observed in quasi-static testing under reversed cyclic loads. Seismic performance of AAC shear-wall structures was evaluated using nonlinear analyses with hysteretic load-displacement behavior based on test results, and using suites of natural and synthetic ground motions from different seismically active regions of the United States.

Using that procedure, values of the factors $R$ and $C_d$ for the seismic design of AAC shear-wall structures are proposed. The proposed value of the factor $(R)$ is the product of the factor $(R_d)$ and an overstrength factor $\Omega_{\text{overstrength}}$, and is equal to 3. The proposed value of $C_d$ is a function of the proposed value of $(R)$ and the overstrength factor $\Omega_{\text{overstrength}}$, and is also equal to 3.

OVERALL SUMMARY AND CONCLUSIONS

The second part of a consistent technical basis for the design of autoclaved aerated concrete (AAC) structures is presented. The basis includes the development and testing of a full-scale, two-story AAC assemblage at The University of Texas at Austin, the development of a rational procedure for the establishment of $R$ and $C_d$ factors for the seismic design of AAC structures, and proposed values for $R$ and $C_d$ based on that procedure.

Acknowledgments

Most of the information reported here is based on a research project on the Seismic Behavior of Autoclaved Aerated Concrete, conducted at The University of Texas at Austin, under the sponsorship of the Autoclaved Aerated Concrete Products Association.

References


<table>
<thead>
<tr>
<th>Base shear at flexural yield, kips (kN)</th>
<th>Critical Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base of Wall</td>
<td>Top of Dowels</td>
</tr>
<tr>
<td>145 (644)</td>
<td>107 (477)</td>
</tr>
<tr>
<td>197 (877)</td>
<td>117 (521)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear wall / Direction of loading</th>
<th>Displacement ductility</th>
<th>Drift ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East wall / south</td>
<td>11.7</td>
<td>0.85</td>
</tr>
<tr>
<td>East wall / north</td>
<td>8.3</td>
<td>0.70</td>
</tr>
<tr>
<td>West wall / south</td>
<td>8.7</td>
<td>0.74</td>
</tr>
<tr>
<td>West wall / north</td>
<td>8.6</td>
<td>0.73</td>
</tr>
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</table>
### Table 3: Drift ratios and displacement ductilities for each wall, with base slip removed

<table>
<thead>
<tr>
<th>Shear wall / Direction of loading</th>
<th>Displacement ductility</th>
<th>Drift ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East wall / south</td>
<td>5.8</td>
<td>0.42</td>
</tr>
<tr>
<td>East wall / north</td>
<td>2.8</td>
<td>0.24</td>
</tr>
<tr>
<td>West wall / south</td>
<td>4.4</td>
<td>0.37</td>
</tr>
<tr>
<td>West wall / north</td>
<td>2.8</td>
<td>0.24</td>
</tr>
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</table>

### Table 4: Physical details and axial load for flexure-dominated walls

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Length, in. (m)</th>
<th>Thickness, in. (m)</th>
<th>Height, in. (m)</th>
<th>Aspect ratio</th>
<th>Axial load, Kips (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>72 (1.8)</td>
<td>8 (0.20)</td>
<td>154 (3.9)</td>
<td>2.1</td>
<td>25 (111)</td>
</tr>
<tr>
<td>14a</td>
<td>56 (1.4)</td>
<td>10 (0.25)</td>
<td>154 (3.9)</td>
<td>2.8</td>
<td>5 (22)</td>
</tr>
<tr>
<td>14b</td>
<td>56 (1.4)</td>
<td>10 (0.25)</td>
<td>154 (3.9)</td>
<td>2.8</td>
<td>5 (22)</td>
</tr>
<tr>
<td>15a</td>
<td>112 (2.8)</td>
<td>10 (0.25)</td>
<td>154 (3.9)</td>
<td>1.4</td>
<td>25 (111)</td>
</tr>
<tr>
<td>15b</td>
<td>112 (2.8)</td>
<td>10 (0.25)</td>
<td>154 (3.9)</td>
<td>1.4</td>
<td>25 (111)</td>
</tr>
<tr>
<td>16</td>
<td>112 (2.8)</td>
<td>10 (0.25)</td>
<td>154 (3.9)</td>
<td>1.4</td>
<td>25 (111)</td>
</tr>
</tbody>
</table>

### Table 5: Maximum drift ratios and displacement ductilities for the flexure-dominated walls

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum drift ratio (%) Loading south</th>
<th>Maximum drift ratio (%) Loading north</th>
<th>Maximum displacement ductility Loading south</th>
<th>Maximum displacement ductility Loading north</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>1.4</td>
<td>1.1</td>
<td>4.78</td>
<td>3.67</td>
</tr>
<tr>
<td>14a</td>
<td>2</td>
<td>--</td>
<td>5.00</td>
<td>--</td>
</tr>
<tr>
<td>14b</td>
<td>2</td>
<td>1</td>
<td>4.84</td>
<td>2.58</td>
</tr>
<tr>
<td>15a</td>
<td>1</td>
<td>1</td>
<td>5.56</td>
<td>5.93</td>
</tr>
<tr>
<td>15b</td>
<td>1</td>
<td>1</td>
<td>4.84</td>
<td>4.84</td>
</tr>
<tr>
<td>16</td>
<td>1</td>
<td>0.4</td>
<td>5.00</td>
<td>1.67</td>
</tr>
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### Table 6: Values of the factor ($R_d$) for different structures and suites of earthquakes

<table>
<thead>
<tr>
<th>Suite of Earthquakes</th>
<th>Structure</th>
<th>Mean $R_d$</th>
<th>Structure</th>
<th>Mean $R_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles</td>
<td>ST-1W-5S</td>
<td>2.37</td>
<td>ST-2W-5S</td>
<td>2.48</td>
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<tr>
<td>Seattle</td>
<td>ST-1W-5S</td>
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<td>2.92</td>
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<tr>
<td>Carbondale</td>
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<td>2.83</td>
<td>ST-2W-5S</td>
<td>3.07</td>
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<tr>
<td>Memphis</td>
<td>ST-1W-5S</td>
<td>2.46</td>
<td>ST-2W-5S</td>
<td>2.66</td>
</tr>
<tr>
<td>Charleston</td>
<td>ST-1W-5S</td>
<td>2.93</td>
<td>ST-2W-5S</td>
<td>2.96</td>
</tr>
<tr>
<td>Los Angeles</td>
<td>ST-1W-3S</td>
<td>1.95</td>
<td>ST-2W-3S</td>
<td>2.19</td>
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<td>Seattle</td>
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<td>2.15</td>
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<td>Carbondale</td>
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<td>ST-2W-3S</td>
<td>3.19</td>
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<tr>
<td>Average</td>
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<td>2.58</td>
<td>Standard</td>
<td>0.35</td>
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<td>Deviation</td>
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<td></td>
<td>10% lower fractile</td>
<td>2.13</td>
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Table 7: Values of the factor ($C_d$) for different structures and suites of earthquakes

<table>
<thead>
<tr>
<th>Suite of Earthquakes</th>
<th>Structure</th>
<th>Mean $C_d$</th>
<th>Structure</th>
<th>Mean $C_d$</th>
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</thead>
<tbody>
<tr>
<td>Los Angeles</td>
<td>ST-1W-5S</td>
<td>4.16</td>
<td>ST-2W-5S</td>
<td>3.74</td>
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<td>Seattle</td>
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<td>3.14</td>
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<td>ST-2W-5S</td>
<td>2.90</td>
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<td>ST-2W-3S</td>
<td>4.44</td>
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<td>Seattle</td>
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<td>3.17</td>
<td>ST-2W-3S</td>
<td>3.63</td>
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<tr>
<td>Carbondale</td>
<td>ST-1W-3S</td>
<td>3.87</td>
<td>ST-2W-3S</td>
<td>4.34</td>
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<tr>
<td>Memphis</td>
<td>ST-1W-3S</td>
<td>3.53</td>
<td>ST-2W-3S</td>
<td>3.51</td>
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<tr>
<td>Charleston</td>
<td>ST-1W-3S</td>
<td>2.87</td>
<td>ST-2W-3S</td>
<td>2.78</td>
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</table>

Average: 3.49
Standard Deviation: 0.54
10% lower fractile: 2.79
10% upper fractile: 4.19

Figure 1: Isometric view of Two-Story AAC Assemblage Specimen

Figure 2: Plan view of horizontal section at base, showing flexural reinforcement and dowels in Two-story AAC Assemblage Specimen
Figure 3: Proposed loading history for Two-story AAC Assemblage Specimen

Figure 4: Overall hysteretic behavior of Two-story AAC Assemblage Specimen

Figure 5: Hysteretic behavior of Two-story AAC Assemblage Specimen (force per story)
Figure 6: Shear span used to calculate the base shear capacity of a wall corresponding to a given flexural capacity

\[ \frac{M}{V} = \frac{H_2 + 2H_1}{2} \]

Figure 7: Example of diagonal cracks and spalling at dowel location

Figure 8: Hysteretic behavior of Two-story AAC Assemblage Specimen with slip removed
Figure 9: Vertical crack at north end of east wall (top and bottom of first story wall)

Figure 10: Cracks in the east wall at the end of the test
Figure 11: Cracks in the west wall at the end of the test

Figure 12: Cracks in the south and north walls at the end of the test
Figure 13  Actual wall element and idealized wall element of CANNY 99

Figure 14  Maximum inelastic displacement and elastic displacements associated with R of 3
Design Examples for Structural Walls and Floor/Roof Panels Constructed of Autoclaved Aerated Concrete (AAC)

by K. Itzler, P.E. and A. Nelson

Synopsis: A general overview of the approach to the design of autoclaved aerated concrete (AAC) structural walls and floor/roof panels is presented. Variations in design approach from concrete and masonry, and design equations specific to AAC are discussed and provided. Design examples illustrate the proposed approach.

Keywords: AAC design; AAC floor/roof panels; AAC walls
I. INTRODUCTION

AAC is a structural product that is manufactured in a variety of different configurations ranging from blocks in a variety of sizes and thicknesses to factory reinforced panels. AAC blocks may be field reinforced similar to conventional masonry units. Panels are factory reinforced with small diameter ASTM-A82 wire, and depending on the configuration of the panels may be field reinforced as well. AAC panels may be used in wall applications or as floor and roof structural elements.

The material specifications for AAC are given in the following ASTM Specifications:


Information about strength class, materials and manufacturing information can be found in these documents.

The design provisions for AAC in the United States have been under development for a number of years. The Masonry Standards Joint Committee (ACI 530) and ACI Subcommittee 523A have both been active in this area with the MSJC concentrating on AAC wall construction, and ACI 523A concentrating on floor and roof panel design. It is anticipated that AAC design provisions will be included in the 2005 MSJC Code, and ACI 523A is developing a design guide for AAC panels. Historically, design provisions for the material have been specified in National Evaluation Reports published by the National Evaluation Service, Inc. as sponsored by various AAC manufacturers. The most recent updates to the design provisions will appear in an ICBO evaluation document sponsored by the Autoclaved Aerated Concrete Producers Association. Proposed design provisions and equations included in this paper are taken from those documents.
II. DESIGN APPROACH

An ultimate strength design approach has been adopted for AAC. Load factors are as specified in ASCE 7-98, “Minimum Design Loads for Buildings and Other Structures”.

The design procedures are as generally set forth in the MSJC Code (particularly Chapter 3) and ACI 318. Several modifications are required to deal with the particular properties of AAC. Design equations that have been proposed are based on extensive testing at the University of Alabama at Birmingham, the University of Texas at Austin as well as several other Universities and commercial laboratories (Construction Technology Laboratories Inc, Skokie, Illinois).

III. NOTATION, DEFINITIONS, DESIGN VALUES

1. AAC Strength Class is defined in ASTM C1386.

<table>
<thead>
<tr>
<th>Strength Class</th>
<th>Minimum Specified Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AAC-2</td>
<td>290</td>
</tr>
<tr>
<td>AAC-4</td>
<td>580</td>
</tr>
<tr>
<td>AAC-6</td>
<td>870</td>
</tr>
</tbody>
</table>

2. While the density of AAC of different strength classes is also specified in ASTM C1386 an increase in the specified densities should be considered due to reinforcing steel, variable moisture content in the field, grout and other considerations. As an example, for an AAC-4 material with a nominal dry density of 37 lb/ft$^3$ as defined per ASTM C1386 a design density of 45 lb/ft$^3$ is recommended.

3. Strength Reduction Factors

   a. Flexure and axial load $\phi = 0.90$
   b. Shear $\phi = 0.80$ or $\phi = .85$ for floor/roof panels
   c. Anchor Pullout $\phi = 0.90$ (AAC breakout)
   d. Development of Splices and Reinforcement $\phi = 0.80$
   e. Bearing $\phi = 0.60$
4. Compressive Strength of AAC - $f_{\text{AAC}}'$.  

$f_{\text{AAC}}'$ is defined by strength class as stated in ASTM C1386. Availability of material varies by manufacturer, however AAC-4 with a $f_{\text{AAC}}' = 580$ psi is a commonly available material.

It should be noted that $f_{\text{AAC}}'$ depends on the strength of the AAC material alone; unlike concrete or clay masonry, the thin bed mortar used in AAC construction exceeds the strength of the AAC units.

5. AAC splitting strength – $f_{i\text{AAC}}$  

$$f_{i\text{AAC}} = 2.4 \sqrt{f_{\text{AAC}}'}$$  

(III-5)

6. AAC modulus of rupture – $f_{r\text{AAC}}$  

$$f_{r\text{AAC}} = 2\left( f_{i\text{AAC}} \right)$$  

(III-6)

$$f_{r\text{AAC}} \leq 50 \text{ psi} \text{ if a section of AAC contains a horizontal leveling bed (ASTM C270 Type M or S mortar)}$$

or

$$f_{r\text{AAC}} \leq 80 \text{ psi if a section of AAC contains a thin bed mortar bed joint.}$$

7. AAC Modulus of Elasticity - $E_{\text{AAC}}$  

$$E_{\text{AAC}} = 6500\left( f_{\text{AAC}}' \right)^{0.6}$$  

(III-7) $f_{\text{AAC}}'$ and $E_{\text{AAC}}$ in psi

8. AAC Shear Modulus - $E_v$  

$$E_v = 0.4 E_{\text{AAC}}$$  

(III-8)

9. Nominal Shear Strength

The nominal shear strength of AAC in wall construction is computed as $V_n = V_{\text{AAC}} + V_s$ similar to that set forth in Chapter 3, Section 3.2.4.1.2 of the 2002 MSJC Code, except $V_{\text{AAC}}$ is substituted for $V_m$ and $f_{\text{AAC}}'$ is substituted for $f_m'$.  

The upset values specified also apply to AAC. Additionally, for walls subject to reversible loads due to wind or seismic loading, only deformed reinforcement embedded in grout shall be counted as shear reinforcement.
10. Nominal Shear Strength of AAC.

a. \( V_{AAC} = 0.95 \sqrt{f'_{AAC}} t \sqrt{1 + \frac{P_u}{2.4 f'_{AAC} l_w t}} \) (111-10-a)

Web shear cracking – fully mortared wall.

b. \( V_{AAC} = 0.66 \sqrt{f'_{AAC}} t \sqrt{1 + \frac{P_u}{2.4 f'_{AAC} l_w t}} \) (111-10-b)

Web shear cracking – unmortared head joints.

c. \( V_{AAC} = 0.9 l_w t \sqrt{f'_{AAC}} A_n + 0.05 P_u \) (III-10-c)

Masonry other than running bond.

11. Nominal shear strength as governed by crushing of diagonal compression strut. (Shear walls)

\[ V_{AAC} = 0.17 (f'_{AAC}) t \left( \frac{h l_w^2}{h^2 + (\frac{3}{4} l_w)^2} \right) \] (III-11) for \( \frac{M_u}{V_u d_v} < 1.5 \)

For walls with \( \frac{M_u}{V_u d_v} \geq 1.5 \) crushing of the diagonal compression strut need not be checked.

12. Nominal shear strength provided by reinforcement

\[ V_s = \left( \frac{A_v}{S} \right) f_y d \] (III-12)

13. Nominal shear strength as governed by out of plane loading.

\[ V_{AAC} = 0.8 \sqrt{f'_{AAC}} b d \] (III-13)

14. AAC direct shear strength

\[ f_v = 0.15 (f'_{AAC}) \] (III-14)
IV. DESIGN ASSUMPTIONS

Design assumptions shall be as set forth in the MSJC Code Section 3.2.2 and ACI 318 Chapter 10, except the maximum useable strain, \( \varepsilon_u \), at the extreme AAC compression fiber shall be 0.003, and the strength of the compression zone shall be calculated as 85 percent of \( f'_{AAC} \) times 67 percent of the compression zone.

V. AAC FLOOR AND ROOF PANELS

Design of AAC Floor and Roof Panels follows the requirements of ACI 318 except for the following:

1. \( \rho_{\text{min}} \) shall be determined as follows:
   \[
   \rho_{\text{min}} = \frac{4 \sqrt{f'_{AAC}}}{f_y} \quad (V-1)
   \]

2. Deflection – The minimum thickness of AAC slabs required to satisfy deflection requirements is determined from Table 9.5(a) of ACI 318 utilizing Footnote (a), and the density of AAC from ASTM C1386. Footnote (b) does not apply. Long term deflections can be accounted for in detailed calculations using an effective modulus of elasticity equal to \( E_{AAC} / 1.5 \cdot (V-2) \)

9.5.2.3 – Immediate 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 \( (M_{cr}, \phi_{cr}) \) and the yielding points \( (M_{yf}, \phi_y) \) on a bilinear moment-curvature diagram. The modulus of rupture shall be as specified in Section III.6.

3. A significant difference between reinforced AAC and reinforced concrete is the mechanism for the development of the factory installed reinforcing steel. Unlike reinforced concrete, where the bond between deformed bars and the concrete develops the tensile strength of the reinforcement, the tensile strength of factory installed reinforcement, smooth ASTM A82 wire, is developed by cross wires welded to the longitudinal reinforcement. The force in the tensile reinforcement is limited by the bearing capacity of the AAC under the cross-wires.

For uniformly loaded panels the following equation has been developed:

\[
V_{a}(L) = 5.1(d)(d_{cross})(l_{cross})f'_{AAC}(\phi)
\]

\[
n_{\text{cross,min}} = \frac{V_{a}(L)}{5.1(d)(d_{cross})(l_{cross})f'_{AAC}(\phi)} \quad (V-3)
\]

where:
\[
d_{cross} = \text{diameter of cross wire}
\]
\[
l_{cross} = \text{length of cross wire}
\]
Autoclaved Aerated Concrete

\[ n_{\text{cross}, \text{min}} = \text{minimum number of equally spaced cross wires to a distance of } L/6 \text{ from support.} \]

\[ \phi = 0.85 \]

**EXAMPLES**

**Bearing Wall – In Plane Loads**

10" AAC block wall

L = 16' wall length

h = 10' wall height

DL = 3.6 k/f dead load

LL = 1.6 k/f live load

#4 @ 24" vertical reinforcement

\[ f'_{AAC} = 580 \text{ psi} \quad \text{AAC compressive strength} \]

\[ f_y = 60000 \text{ psi} \quad \text{steel strength} \]

\[ f_g = 3000 \text{ psi} \quad \text{grout strength} \]

\[ d_{\text{core}} = 3 \text{ in} \quad \text{core diameter} \]

\[ \phi = 0.9 \quad \text{flexure and axial load strength reduction factor} \]

\[ W_{AAC} = 45 \text{ pcf} \quad \text{wall density} \]

Wall design for plane loads follows Section 3.2.6-MSJC

1. Wall loads

\[ \text{DL} = 3.6 \text{ k/f} \]

\[ \text{LL} = 1.6 \text{ k/f}. \]

2. Load combinations

Assume 2ft wide design strip

\[ P_u = 1.2DL + 1.6LL \]

\[ = 1.2 \cdot \left(3.6 \cdot 2 + \frac{45}{1000} \cdot \frac{10}{12} \cdot 2\right) + 1.6 \cdot 1.6 \cdot 2 \]

\[ = 13.85k \]

3. Determine nominal axial compressive strength

Calculate radius of gyration.

\[ r = \sqrt{\frac{l}{A}} = \frac{t}{\sqrt{12}} = \frac{10}{\sqrt{12}} = 2.88 \]
According to Section 3.2.4.1.1 – (a) formula 3-16 will be applied.

\[ P_n = 0.8[0.85 f_{AAC} (A_n - A_s) + f_s A_s ](1 - (\frac{h}{140r})^2) \]  

\[ A_n = b \cdot t = 240 \text{ in}^2 \]

As in the design of conventional masonry, untied compression steel may not be counted as contributing to compression resistance.

\[ P_n = 86.27K \]
\[ P_n < \phi P_n, \quad \phi = 0.9 \]
\[ 13.85' < 77.64K \quad \text{O.K.} \]

Use \( t = 10' \) thick AAC wall with #4@24 centered in wall.

**Wall Design For Out Of Plane Loads**

8" AAC block wall

- L = 8' wall length
- h = 10' wall height
- DL = 2 k/f dead load
- LL = 0.8 k/f live load
- w = 24.2 psf wind load
- #4 @ 24” vertical reinforcement

- \( f'_{AAC} = 580 \text{ psi} \) AAC compressive strength
- \( f_y = 60000 \text{ psi} \) steel strength
- \( f_g = 3000 \text{ psi} \) grout strength
- \( d_{core} = 3 \text{ in} \) core diameter
- \( \mu = 1 \) sliding coefficient of friction at leveling bed joint
- \( \phi_{shear} = 0.8 \) shear strength reduction factor
- \( \phi_{bending} = 0.9 \) bending strength reduction factor
- \( E_s = 29000000 \text{ psi} \)
- \( w_{AAC} = 45 \text{ psi} \) wall density for design
1. Wall loads
   Consider 2ft wide design strip
   Axial Loads: \( DL = 2k/f \)
   \( LL = 0.8k/f \)

   \[ DL_{wall} = \frac{45}{1000} \cdot \frac{8''}{12} = 0.03k/ft^2 \]

   Lateral Loads – Wind Loads

   \( w = 24.2\, psf \)

2. Load Combinations

   Case I 1.2 DL + 1.6LL + 0.8 W.

   \[ P_{w1} = 1.2DL + 1.6LL \]
   \[ = 1.2 \cdot 2 \cdot 2 + 1.6 \cdot 0.8 \cdot 2 \]
   \[ = 7.36k \]

   \( w_{w1} = 0.8W \)

   \[ = 0.8 \left( \frac{24.2}{1000} \right) \cdot 2' \]

   \[ = 0.038k/ft \]

   Case II 1.2 DL + 0.5LL + 1.6 W

   \[ P_{w2} = 1.2DL + 1.5LL \]
   \[ = 1.2 \cdot 2 \cdot 2 + 0.5 \cdot 0.8 \cdot 2 \]
   \[ = 5.6k \]

   \( w_{w2} = 1.6W \)

   \[ = 1.6 \left( \frac{24.2}{1000} \right) \cdot 2' \]

   \[ = 0.077k/ft \]
3. Determine Nominal Shear Strength

Nominal Shear Strength is determined according to (III–13) of this paper.

\[ V_{AAC} = 0.8 \cdot \sqrt{\frac{f_{u,AAC}}{psi}} \cdot b \cdot d = 1.85K \]

\[ V_u = \max(w_{u1}, w_{u2}) \cdot \frac{h}{2} \]

\[ V_u = 0.385K \]

Therefore:

\[ Vu \leq \phi_{shear} V_{AAC} \]

\[ 0.385K \leq 1.48K \quad O.K. \]

4. Check sliding

\[ V_{AAC} = \mu P_{u,DL}, \quad \mu = 1.0 \]

\[ P_{u,DL} = \max(P_{u,f1} + P_{u,f2}) + 1.2P_{wall} \]

\[ = 6.32K \]

\[ V_{AAC} = 6.32K \]

\[ V_u \leq 5.05 \]

\[ 0.385K \leq 5.05 \quad O.K. \]

5. Walls with factored axial stress of 0.05f′m or less – Section 3.2.5.4 (MSJC).

The procedures set forth in this section will be used when the factored axial load stress at the location of maximum moment satisfies the requirement computed by equation (3-23)-MSJC.

\[ \frac{P_u}{A_g} \leq 0.05 f\prime_m \]

\[ P_u = \max(P_{uf1}, P_{uf2}) + P_w = 8.08K \]

\[ A_g = b \cdot t = 192in^2 \]

\[ \frac{P_u}{A_g} = 42.08psi \]

\[ 42.08psi \leq 29psi \quad not \ good \]

Therefore follow section 3.2.5.5.

6. Walls with factored axial stress greater than 0.05f′m – Section 3.2.5.5 – MSJC.

The procedures set forth in this section shall be used for the design of masonry walls where the factored axial load stress at the location of the maximum
moment exceeds $0.05f'_m$. These provisions shall not be applied to walls with factored axial load stress equal to or exceeding $0.2f'_m$ or slenderness ratios exceeding 30. Such walls shall be designed in accordance with the provisions of Section 3.2.5.4 and shall have a minimum nominal thickness of 6 in.

\[
\left( \frac{P_u}{A_g} \right) \geq 0.05f'_m
\]

42.08 psi $\geq$ 29 psi \hspace{1cm} O.K.

42.08 psi $\geq$ 0.2$f'_m$

42.08 psi $\geq$ 116 psi \hspace{1cm} NOT TRUE

Therefore the wall shall be designed in accordance with Section 3.2.5.4 – Walls with factored axial stress of $0.05f'_m$ or less.

7. Design in accordance with Section 3.2.5.4 – Walls with factored axial stress of $0.05f'_m$ or less. Factored moment and axial force shall be determined at the mid height of the wall and shall be used for design. The factored moment $M_u$ is given by equation (3-24)-MSJC.

\[
M_u = \frac{w_t h^2}{8} + P_{uf} \frac{e_u}{2} + P_u \delta_u
\]

\[
P_u = P_{uw} + P_{uf}.
\]

The two cases shall be considered.

**Case I**

\[1.2DL + 1.6LL + 0.8W\]

\[P_{u1} = P_{uf1} + P_{uw}\]

\[P_{u1} = 8.08^k\]

\[w_{u1} = 0.038^k/ft\]

Assume $\delta_u = 0.05$ in

\[M_{u1} = \frac{w_{u1} h^2}{8} + P_{uf1} \frac{e_u}{2} + P_{u1} \delta u\]

\[= 0.508K - ft. = 508lb. ft.\]

*From 3.2.5.6 MSJC*
\[
\delta_u = \begin{cases} 
M_{u1} < M_{cr}, \delta_u = \frac{5M_{u1}h^2}{48E_{AAC}I_g} \\
M_u < M_{u1} < M_n, \delta_u = \frac{5M_{cr}h^2}{48E_{AAC}I_g} + \frac{5(M_{u1} - M_{cr})h^2}{48E_{AAC}I_{cr}} 
\end{cases}
\]

\[M_{cr} = S_n(f_{rAAC} + \frac{P_{u1}}{A_n}) \quad \text{Similar to (3–32) MSJC}\]

\[f_{rAAC} = 2f_{AAC}\]

\[f_{AAC} = 2.4 \sqrt{\frac{f_{AAC}}{psi}} \quad \text{psi}\]

\[= 2.4\sqrt{580} = 57.79 \text{ psi}\]

\[f_{rAAC} = 115.6 \text{ psi} > 80 \text{ psi} \quad \text{due to thin set mortar joint use 80 psi}\]

\[S_n = \frac{bt^2}{6}\]

\[= 256 \text{ in}^3\]

\[M_{cr} = 2604.44 \text{ lb-ft}\]

\[M_{u1} < M_{cr}\]

\[508 \text{ lb-ft} < 2604.44 \text{ lb-ft}\]

\[\delta_u = \frac{5M_{u1}h^2}{48E_{AAC}I_g}\]

\[E_{AAC} = 6500 \cdot (f'_{ACC})^{0.6}\]

\[= 295781 \text{ psi}\]

\[I_g = \frac{bt^3}{12}\]

\[= 1024 \text{ in}^4\]

\[\delta_u = 0.03 \text{ in} < 0.05 \text{ in}\]

Therefore assumption O.K.
Case II

1.20L + 0.5LL + 1.6W

\[ P_{u2} = P_{uf2} + P_{uw} = 6.32K \]

\[ w_{u2} = 0.077 \, \text{k/ft} \]

Assume \( \delta = 0.06 \text{ in} \)

\[ M_{u2} = \frac{w_{u2} h^2}{8} + P_{uf2} \frac{e_u}{2} + P_{u2} \delta_u \]

\[ = 994 \text{ lb} - \text{ ft} \]

\[ \delta_u = \begin{cases} 
M_{u2} < M_{cr}, & \delta_u = \frac{5M_{u2} h^2}{48E_{ACC} I_g} \\
M_{cr} < M_{u2} < M_n, & \delta_u = \frac{5M_{cr} h^2}{48E_{ACC} I_g} + \frac{5(M_{u2} - M_{cr}) h^2}{48E_{ACC} I_{cr}} 
\end{cases} \]

\[ M_{cr} = S_n \left( f_{r,AAC} + \frac{P_{u2}}{A_n} \right) \]

\[ = 2408.9 \text{ lb} - \text{ ft} \]

\[ M_{u2} < M_{cr} \]

994 lb – ft < 2408.91 lb – ft

\[ \delta_u = \frac{5M_{u2} h^2}{48E_{ACC} I_g} \]

\[ = 0.059 \text{ in} \]

Therefore assumption O.K.

The factored moment \( M_u \) to be used is given by -

\[ M_u = \max \left( M_{u1}, M_{u2} \right) \]

\[ = 994 \text{ lb} - \text{ ft} \]
The design strength for out-of-plane wall loading shall be in accordance with Equation (3-26) (MSJC):

\[ Mu < \phi_{bending} M_n \quad (3 - 26) \text{ MSJC} \]

\[ M_n = \left( A_s \cdot f_y + P_u \right) \left( d - \frac{a}{2} \right) \]

\[ P_u = \min \left( P_{u1}, P_{u2} \right) \]

\[ = 6.32K \]

\[ a = \frac{P_u + A_s f_y}{0.85 f'_{AAC} b} \]

\[ a = 1.54in \]

\[ M_n = \left( A_s f_y + P_u \right) \left( d - \frac{a}{2} \right) \]

\[ = 4931.13 lb - ft \]

\[ Mu \leq \phi_{bending} M_n \quad \phi_{bending} = 0.9 \]

\[ 994 lb - ft \leq 4438.02 lb - ft \quad \text{O.K.} \]

Check reinforcing bar size limitations:

\[ A_{\text{grout}} = \pi \cdot \left( \frac{d_{\text{core}}}{2} \right)^2 \]

\[ = \pi \cdot \left( \frac{3}{2} \right)^2 \]

\[ = 7.06 \text{ in}^2 \]

\[ \frac{A_s}{A_{\text{grout}}} \leq 3\% \quad \text{Limitation of 4.5\% in non plastic hinge area is an} \]

\[ \frac{0.20}{7.06} = 2.8\% \leq 4.5\% \quad \text{O.K.} \]

exception from MSJC.
8. **DEFLECTION DESIGN** – Design procedure follows MSJC Code section 3.2.5.6 except for the formula used to determine $M_u$.

Maximum deflection of wall is given by relation:

$$\delta_s \leq 0.007h$$  
see eq. (3-29) MSJC

$$\delta_s \leq 0.007 \cdot 10 \cdot 12$$

$$\delta_s \leq 0.84\text{ in}$$

Determine $M_{ser}$

$$M_{ser} = w_{ser} \frac{h^2}{8}$$

$$= 605\text{ lb} - \text{ ft}$$

Determine $M_{cr}$

$$M_{cr} = S_n \left( f_{r, AAC} + \frac{P}{A_n} \right)$$

$$P = (DL + LL + DL_{wall}) \times 2'$$

$$= 6.2K = 6200\text{ lb}$$

$$M_u = S_n \left( f_{r, AAC} + \frac{P}{A_n} \right)$$

$$= 2395.55\text{ lb} - \text{ ft}$$

Therefore $M_{ser} < M_{cr}$

$$605\text{ lb} - \text{ ft} \leq 2395.55\text{ lb} - \text{ ft} \quad \text{O.K.}$$

Determine deflection

$$\delta_s = \frac{5M_{ser}h^2}{48 \cdot E_{AAC}I_g}$$  
see equ. (3-30) MSJC

$$= 0.035\text{ in}$$

$$\delta_s < \delta_{max}$$

$$0.035\text{ in} < 0.84\text{ in}$$ therefore deflection O.K.

Use $t=8''$ thick AAC wall with #4@24 centered in wall.

**Design Of Shear Wall**

$h = 10'$  
wall height

$t = 8''$  
wall thickness
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$l_w = 12.25'$  wall length

$w = 45$ pcf  wall density

$f_y = 60000$ psi  tensile strength of steel

$A_s = 0.44in^2$  tension reinforcing (#5 bar)

$f'_{AAC} = 580$ psi  AAC compressive strength

$\mu = 1.0$  at leveling bed joint

$\phi_{shear} = 0.8$  shear strength reduction factor

$\phi_{bending} = 0.9$  bending

$c_c = 4$ in  distance from edge of wall to tension reinforcing

$A_v = 1in^2$  vertical shear reinforcement

$d = lw - c_c$  distance from the compression zone to the tension steel

$A_h = 0.4in^2$  horizontal reinforcement

The loads used in this example are as follows:

$P_u = 6.96K, \ M_u = 295.85K - ft, \ V_u = 35.84K$
A. Determine interaction diagram

1. Determine $P_b$, $M_b$, at the balanced point

Balanced strain condition:

$$
\frac{c_b}{d} = \frac{0.003}{f_y' + 0.003}
$$

$c_b$ – neutral axis distance for the balanced strain condition.

$$
d = lw - c_c = 143''
$$

$$
c_b = 84.63'' = 85''
$$

$$
a_b = c_b \beta_1 = 85 \cdot 0.67 = 57''
$$

$$
P_b = 0.85 f'_{AAC} a_b t - A_s f_y
$$

$$
= 198408 \text{lbs} \quad \rightarrow \phi P_b = 178.56^k
$$

$$
M_b = P_b \left(\frac{lw}{2} - \frac{ab}{2}\right) + A_s f_y \left(d - \frac{lw}{2}\right)
$$

$$
= 11951160 \text{in-lbs} \quad \rightarrow \phi M_b = 0.9 \cdot \frac{11951160}{12000} = 896.34 \text{ ft-k}
$$

$$
e_b = M_b / P_b = 60.23''
$$

2. Determine $P_o$ – concentric load only determine $h/r$ ratio

$$
\frac{h}{r} = \frac{h}{t} = \frac{h\sqrt{12}}{t} = 51.96 < 99
$$

$$
\frac{\sqrt{12}}{t}
$$
According to 3.2.4.1.1 MSJC use formula (3-16) to determine nominal axial load strength \( P_n \)

\[
P_{n,\text{max}} = 0.8(0.85 f'_{AAC} (A_n - A_s) + f_y A_s)(1 - \left(\frac{h}{140r}\right)^2)
\]

\[
P_{n,\text{max}} = 0.8 \cdot [0.85 \cdot 580 \cdot 1176][1 - \left(\frac{51.96}{140}\right)^2]
\]

\( A_n = 1176 \)

\( P_n = 399928.34 \text{ lbs} \)

\( \phi P_n = 360K \)

3. Determine \( M_o \) – pure bending

\[
a = \frac{A_s f_y}{0.85 \cdot f'_{AAC} \cdot t}
\]

\( = 6.69 \text{ in} \)

\[
M_o = A_s \cdot f_y \cdot (d - \frac{a}{2})
\]

\( = 3686892 \text{ in-lb} \)

\( \phi M_o = 276.52 \text{ ft-lb} \)

4. Compression control \( c > c_b \)

a) Take \( c = 135 \text{ in} \)

\( a = 0.67 \cdot 135 = 90.45 \text{ in} \)

\[
f_s = 0.003 \cdot 29000 \cdot \frac{d - c}{c}
\]

\( = 5.15 \text{ KSI} \)

\( P_n = 0.85 f'_{AAC} at - A_s f_s = 354468.8 \text{ lbs} \)

\( \phi P_n = 319K \)

\[
M_n = 0.85 \cdot f'_{AAC} at(l_w - a) / 2 + A_s f_s (d - l_w / 2)
\]

\( = 10244163.47 \text{ in-lbs} \)

\( \phi M_n = 768.31 \text{ ft-lb} \)

\( e = M_n / P_n = 28.9 \text{ in} \)
b) Take \( c = 100 \) in
\( a = 67 \) in

\[
M_n = 0.85 \cdot f'_{AAC} at(l_w - a) / 2 + A_s f_y (d - l_w / 2)
\]

\[
= 11713917.8 \text{in} - \text{lbs}
\]

\[
\phi M_n = 878.54 \text{ft} - k
\]

\[
e = M_n / P_n = 42.27
\]

5. Tension Control \( c < c_b \)

Take \( c = 60 \) in

\[
a = 40.2 \text{in}
\]

\[
P_n = 0.85 f'_{AAC} ta - A_s f_y
\]

\[
= 132148.8 \text{lbs}
\]

\[
\phi P_n = 118.93K
\]

\[
M_n = 0.85 f'_{AAC} \left( l_w - a \right) / 2 + A_s f_y \left( d - l_w / 2 \right)
\]

\[
= 10301305.92 \text{ft} - \text{lbs}
\]

\[
\phi M_n = 772.6 \text{ft} - k
\]

\[
e = \frac{M_n}{P_n} = 77.95''
\]

6. Determine \( P_n, M_n \) when \( 0.10 f'_{AAC} A_g = \phi P_n \)

\[
0.10 f'_{AAC} A_g = 0.1 f'_{AAC} l_w t = 68208 \text{lbs}
\]

\[
\phi P_n = 68208 \text{lbs} \quad P_n = 75786.66 \text{lbs}
\]

\[
P_n = 0.85 f'_{AAC} ta - A_s f_y
\]

\[
= 75786.66 \text{lbs}
\]

\[
a = 25.91 \text{in}
\]

\[
c = \frac{a}{0.67} = 38.67 \text{in}
\]

\[
M_n = 0.85 f'_{AAC} ta\left( l_w - a \right) / 2 + A_s f_y \left( d - l_w / 2 \right)
\]

\[
= 8021835.43 \text{in} - \text{lbs}
\]
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\[ \phi M_n = 601.64 \text{ ft} - k \]

\[ e = \frac{M_n}{P_n} = 105.84'' \]

<table>
<thead>
<tr>
<th>( \phi P_n (k) )</th>
<th>( \phi M_n (\text{ft} - K) )</th>
</tr>
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<tr>
<td>360</td>
<td>0</td>
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<td>319</td>
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</tr>
<tr>
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<td>896.34</td>
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<td>118.93</td>
<td>772.6</td>
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<tr>
<td>68.21</td>
<td>601.64</td>
</tr>
<tr>
<td>0</td>
<td>276.52</td>
</tr>
</tbody>
</table>

**SHEAR WALL**

**INTERACTION P-M DIAGRAM**
B. Determine nominal masonry shear strength as governed by web-shear cracking. Nominal masonry shear strength as governed by web-shear cracking, \( V_m \), shall be computed using equation from (III-10-a) of this paper. Equation (III-10-a) applies for AAC masonry with mortared head joints.

\[
V_{AACwsc} = 0.95l_w t \sqrt{\frac{f'_{AAC}}{psi}} \left[ 1 + \frac{P_u}{2.4} \frac{f'_{AAC}}{psi} l_w t \right]
\]

\( V_{AACwsc} = 28.25K \)

C. Determine nominal shear strength as governed by crushing of diagonal compressive strut. For walls with \( \frac{M_u}{V_u d} < 1.5 \), nominal shear strength, \( V_{AAC} \), shall as governed by crushing of a diagonal strut, shall be computed as follows:

\[
\frac{M_u}{V_u d} = 0.69
\]

\( 0.69 < 1.5 \)

\[
V_{AACcds} = 0.17 f'_{AAC} t h^2 w \frac{h l^2 w}{h^2 + (0.75 l_w)^2}
\]

\( V_{AACcds} = 77.02K \)

Therefore \( V_{AAC} = \min(V_{AACwsc}, V_{AACcds}) \)

\( = 28.25K \)

D. Determine nominal shear as governed by sliding shear. At a mortared head joint by:

\[
V_{AAC} = \phi \left[ \mu P_u + \mu (A_y + A_z \cdot 2) f_y \right]
\]

\( = 95.8 > V_{u\max} = 35.84K \)

E. Nominal shear strength provided by shear reinforcement

\[
V_s = \left( \frac{A_h}{s} \right) f_y d = 71.5K
\]
F. Determine nominal shear strength

Nominal shear strength shall be computed using equation (3-18) and either equation (3-19) or equation (3-20) from MSJC as appropriate:

\[ V_n = V_m + V_s \]  \hspace{1cm} (3-18)

\[ \frac{M_u}{V_u d_v} = 0.69 \]

\[ 0.25 < 0.69 < 1 \]

Therefore the maximum value of \( V_n \) for \( \frac{M_u}{V_u d_v} \) between 0.25 and 1 may be interpolated.

(a) \[ V_n \leq 6A_n \sqrt{f'_{AAC}} \quad \frac{M_u}{V_u d_v} \leq 0.25 \]

\[ A_n = l_w \cdot t = 1176 \]

\[ V_n = 169.93K \]

(b) \[ V_n \leq 4A_n \sqrt{f'_{ACC}} \quad \frac{M_u}{V_u d_v} > 1 \]

\[ V_n = 113.28K \]

\[ V_{n_{\text{max}}} = 136.7K \quad \text{obtained from interpolation.} \]

\[ V_n = V_{AAC} + V_s = 88.25 \]

Nominal masonry shear strength shall be taken as the least of the values computed in sections C and E:

\[ \phi V_n = \min(\phi_{\text{sliding}} V_{AAC_{\text{sliding}}}, \phi_{\text{shear}} V_n) \]

\[ = 70.6K \]
Check to see if design shear strength $\phi V_n$ exceeds the shear corresponding to the development of $1.25 M_n$, except that the nominal shear strength ($V_n$) need not exceed 2.5 times required shear strength ($V_u$).

$$V_{u1.25} = \frac{1.25 M_n}{M_u} = 52.28 K$$

$$P_u = \phi P_n = 0.85 f'_{AAC} t a - A_s f_y$$

$$a = 8.45"$$

$$M_n = 0.85 f'_{AAC} t a \left( \frac{l_w - a}{2} \right) + A_s f_y \left( a - \frac{l_w}{2} \right)$$

$$= 345.29 ft-K$$

$$V_{u1.25} = 52.28 K \leq \phi V_n = 70.6 K \quad O.K.$$
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$DL = 58 \text{ psf}$

$LL = 100 \text{ psf}$

Use wires $\phi = 8\text{mm}$

1. Determine the reinforcement

$w = 1.2 DL + 1.6 LL$

$= \left(1.2 \cdot 58 + 1.6 \cdot 100\right)^2$

$= 460 \text{ psf}$

Maximum moment between supports $M_{@7.73} = 13.78 \text{ ft-k}$

$M_{@\text{support}} = 2.07 \text{ ft-k}$

Therefore $M_u = 13.78 \text{ ft-k}$

$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right)$

$a = \frac{A_s f_y}{0.85 f_{AAC}' b}$

$M_u = \phi A_s f_y \left(d - \frac{A_s f_y}{2 \cdot 0.85 f_{AAC}' b}\right)$

$165360 = 0.9 \cdot A_s \cdot 80000 \cdot 9 - \frac{0.9 A_s^2 \cdot 80000^2}{2 \cdot 0.85 \cdot 580 \cdot 24}$

$d = 10 - 1 = 9 \text{ in}$

$165360 = 648000 A_s - 243407.71 A_s^2$

$243407.71 A_s^2 - 648000 A_s + 165360 = 0$

$A_s = 0.285 in^2$
Take 8mm bar \[ A_{\phi8mm} = \frac{\pi d^2}{4} = \frac{\pi \cdot 0.31^2}{4} = 0.075in^2 \]

⇒ Provide 4 \( \phi8mm \) bars. \( A_s = 0.3in^2 \)

**Check:**

\[ \phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) \]

\[ a = \frac{A_s f_y}{0.85 f'_{AAC} b} = \frac{0.3 \cdot 80000}{0.85 \cdot 580 \cdot 24} = 2.02in \]

\[ \phi M_n = 0.9 \cdot 0.3 \cdot 80000 \cdot \left( 9 - \frac{2.02}{2} \right) \]

\[ = 172584in - lbs = 14.4\ ft - k > M_u \]

2. **Determine** \( \delta_{\text{min}} \)

\[ \rho_{\text{min}} = \frac{4\sqrt{f'_{AAC}}}{f_y} = \frac{4\sqrt{580}}{80000} = 0.0012 \]

\[ \rho = \frac{A_i}{bd} = \frac{0.3}{24 \cdot 9} = 0.0013 > \rho_{\text{min}} \quad O.K. \]

(V-1)

3. **Check shear capacity**

Shear force at distance \( d \) from the face of the interior support

\[ b = 9 + \frac{8}{2} = 13\ in \]

\[ V_u = 3.56 - 0.46 \cdot \frac{13}{12} = 3.06K \]

Shear capacity is given by formula:

\[ V_{AAC} = 0.8 \sqrt{f'_{AAC} bd} \]

\[ = 4161.56\ lbs \]

\[ \phi V_{AAC} = 3329.26\ lbs = 3.33K \]

\[ V_u = 3.06K < \phi V_{AAC} = 3.33K \quad O.K. \]
4. Check deflection

For AAC slabs the minimum thickness $t$ is given by equation (V-2):

$$t = \frac{1}{24} \left(1.65 - 0.005 w\right)$$

$$= \frac{(16 \cdot 12 - 8)}{24} \left(1.65 - 0.005 \cdot 37\right) = 11.23\text{in} > 10\text{in}$$

Therefore need to check deflection

Determine cracked moment $M_{cr}$

$$E_{AAC} = 6.5 \left(f_{AAC}' \right)^{0.6} = 6.5 \left(580\right)^{0.6} = 295.78\text{ksi} \quad \text{elastic modulus}$$

$$E_{AAC}' = \frac{E_{AAC}}{1.5} = \frac{295.78}{1.5} = 197.19\text{ksi} \quad \text{reduced elastic modulus for long term deflection}$$

$$f_{i_{AAC}} = 2.4 \sqrt{f_{AAC}'} = 2.4 \sqrt{580} = 57.8\text{psi} \quad \text{see (III - 7)}$$

$$f_{r_{AAC}} = 2 \left(f_{i_{AAC}}\right) = 2 \cdot 57.8 = 115.6\text{psi} \quad \text{see (III - 6)}$$

$$\eta = \frac{E_s}{E_{AAC}'} = \frac{29000}{197} = 147.2 \quad \text{modular ratio}$$

$$I_g = \frac{bt^3}{12} + 2A_f \left(\frac{t - 2c_c}{2}\right)^2 \quad \text{moment of inertia of the gross concrete cross section}$$
\[ A_{st} = \eta A_s = 147.2 \cdot (0.075 \cdot 4) = 44.16 \quad \text{transformed area of steel in the AAC} \]

\[ I_g = \frac{24 \cdot 10^3}{12} + 2 \cdot 44.16 \left( \frac{10 - 2 \cdot 1}{2} \right)^2 = 3413.12 \text{in}^4 \]

\[ M_{cr} = \frac{f_{r AAC} I_g}{t} = \frac{115.6 \cdot 3413.12}{10} = 78911.33 \text{in} - \text{lbs} = 6.57 \text{ft} - K \]

Determine maximum unfactored moment.

\[ w = (58 + 100) \cdot 2 = 316 \text{ psf} \]

\[ M_{\text{max}} = 9.64 \text{ ft} - K \]

Maximum unfactored moment \( M = 9.64 \text{ ft} - K > M_{cr} = 6.57 \text{ ft} - K \)
Therefore yielding moment needs to be calculated.

Determine yielding moment

\[ \eta A_y (9 - x) - \rho A_y (x - 1) - \frac{bx^2}{2} = 0 \]

\[ 44.16(9 - x) - 44.16(x - 1) - \frac{24x^2}{2} = 0 \]

\[ 12x^2 + 88.32x - 441.6 = 0 \quad \Rightarrow \quad x = 3.41'' \]

\[ I_{cr} = \frac{24 \cdot 3.41^3}{3} + 44.16(3.41 - 1)^2 + 44.16(9 - 3.41)^2 = 1953.62in^4 \]

\[ M_y = E'_{AAC} \left( \frac{F_y}{E_s} \frac{I_{cr}}{d - x} \right) \]

\[ = 197.19 \cdot 10^3 \cdot 1953.61 \cdot \left( \frac{80000}{29000 \cdot 10^3} \frac{9 - 3.41}{9 - 3.41} \right) \]

\[ = 190109.114in - lb = 15.84ft - K \]
Determine curvature at cracking, yielding and corresponding to maximum unfactored moment.

$$\phi_{cr} = \frac{M_{cr}}{E'_{AAC}I_g}$$

$$= \frac{78911.33}{197.19 \cdot 10^3 \cdot 3413.12}$$

$$= 0.000117 \text{ in}^{-1}$$

$$\phi_y = \frac{M_y}{E'_{AAC}I_{cr}}$$

$$= \frac{190109.114}{197.19 \cdot 10^3 \cdot 1953.62}$$

$$= 0.00049 \text{ in}^{-1}$$

Interpolate in order to determine $\phi$ corresponding to maximum unfactored moment.

0.000117 ........ 78911.33

$\phi = ?$  115680  $\phi = 0.00024$

0.00049 ........ 190109.114

Determine long term deflections.

$$E'_{AAC}I_e = \frac{M}{\phi} = \frac{115680}{0.00024} = 482000000 \text{ lb in}^2$$

$$\delta_{\text{long-term}} = \frac{wx}{24EI\ell} \left(t^4 - 2t^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2\right)$$

$$\delta_{\text{long-term}} = 0.87 \text{ in}$$

$$\delta_{\text{max}} = \frac{l}{240} = 0.8 \text{ in} \quad \text{Deflection is not O.K.}$$

Therefore either the thickness of the panel must be increased or the area of reinforcement must be increased.
5. Anchorage design

\[ \frac{l}{6} = \frac{16 \times 12}{6} = 32 \text{in} \]

\[ V_{\text{max}} = \frac{V_u}{\phi} = \frac{5.18 \times 10^3}{0.8} = 6475 \text{lbs} \]

\[ n_{\text{cross min}} = \frac{V_{\text{max}} l}{5.1 \cdot d \cdot d_{\text{cross}} \cdot l_{\text{cross}} \cdot f'_{\text{AAC}}} \]

\[ = \frac{6475 \cdot (16 \times 12)}{5.1 \cdot 9 \cdot 0.31 \cdot 22 \cdot 580} = 6.84 \approx 7 \]

\[ d_{\text{cross}} = \delta_{\text{min}} = 0.31 \text{in} \]

\[ l_{\text{cross}} = (24" - 2") = 22 \text{in} \]

Use 7 φ8mm cross wires over first 32in

\[ S_{\text{min}} = \frac{31}{7} = 4.42 \approx 4 \text{in} \]

Use 8mm wires @ 4in for the first 32in of the panel. At the continuous support use 8mm wires @ 4in on both sides of the support and for the entire cantilever. For the rest of the panel use 8mm wires @ 8in., i.e. twice required spacing.

**CONCLUSION**

AAC walls and floor/roof construction can be designed utilizing the requirements set forth in the MSJC Code and ACI 318, respectfully, with certain specific modifications related to Autoclaved Aerated Concrete. Walls may be used as load bearing walls and shear walls, for low and mid-rise buildings or as cladding in any type of structure. AAC floor and roof panels are suitable for spans up to twenty feet in residential, commercial or institutional type building construction.

**REFERENCES**

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LIST OF NOTATIONS

\( A_s = \) area of steel
\( d = \) distance from the compression zone to tension steel
\( d_{\text{core}} = \) core diameter
\( d_{\text{cross}} = \) diameter of crosswire
\( E_{\text{AAC}} = \) AAC modulus of elasticity
\( E_v = \) AAC shear modulus
\( f'_{\text{AAC}} = \) compressive strength of AAC
\( f_{\text{rAAC}} = \) AAC modulus of rupture
\( f_{\text{IAAC}} = \) AAC splitting strength
\( f_g = \) grout strength
\( f_v = \) AAC direct shear strength
\( f_y = \) yield strength of steel
\( h = \) wall height
\( I_{cr} = \) cracked moment of inertia
\( I_g = \) moment of inertia of gross concrete cross section
\( l_{\text{cross}} = \) length of crosswire
\( M_b = \) moment at balanced point
\( M_{cr} = \) cracking moment
\( n_{\text{cross, min}} = \) minimum number of equally spaced cross wires to a distance of L/6 from support
\( P_b = \) axial load at balanced point
\( r = \) radius of gyration
\( t = \) thickness of wall
\( V_{\text{AAC}} = \) nominal shear strength of AAC
\( V_n = \) nominal shear strength
\( V_s = \) nominal shear strength provided by reinforcement
\( \rho_{\text{min}} = \) minimum reinforcement ratio
\( \delta = \) deflection
Properties of Fiber-Reinforced Lightweight Concrete

by C. Shi, Y. Wu and M. Riefler

Synopsis: The use of lightweight concrete has many advantages over conventional concrete. The reduced self-weight of lightweight concrete will reduce gravity load and seismic inertial mass. The lightweight concrete reported here has compressive strengths from 8 to 50 MPa with dry densities from 800 to 1400 kg/m$^3$, which is strong enough for any load-bearing and non-load-bearing applications. The compressive strength to flexural strength ratio increases as the compressive strength of the concrete increases. The introduction of a small amount of fiber does not affect the flexural strength and drying shrinkage of the concrete, but improves the ductility and handling properties of the product very significantly. The lightweight concrete has a higher moisture loss during drying, but a lower shrinkage than the normal weight concrete due to the buffer effect of the moisture in the lightweight aggregate. Properly designed fiber-reinforced ultra lightweight concrete can be easily cut, sawed and nailed like wood.

Keywords: density; drying shrinkage; ductility; fiber; lightweight concrete; strength
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Monte Riefler was the president of Advanced Materials Technologies LLC and passed away in March 2003.

INTRODUCTION

Lightweight concrete has been used for a number of applications and is also known for its good performance and durability [1]. In structural applications, the self-weight of the concrete structure is important since it represents a large portion of the total load. The reduced self-weight of lightweight concrete will reduce gravity load and seismic inertial mass, resulting in reduced member size and foundation force. Use of lightweight concrete can be of interest in retrofit applications where a concrete column jacket is desired due to architectural reason over other method such as steel or composite jackets. In that situation, a normal weight concrete jacket might result in foundation forces that would require expensive foundation retrofit where as a lightweight concrete jacket may not require a footing retrofit [2]. The use of lightweight aggregate concrete in a structure is usually predicated on lower overall costs [1]. While lightweight concrete may cost more per cubic yard than normal weight concrete, the structure may cost less as a result of reduced dead weight and lower foundation costs.

There are two ways to produce lightweight concrete: introduction of air into aggregates (use of lightweight) and introduction of air bubbles into cement pastes. Autoclaved aerated concrete (AAC) is a great example of introduction of air into cement paste. It uses aluminum powders to generate uniform micro-bubbles within concrete to produce a lightweight concrete. AAC has been around for more than 70 years and usually has strength of less than 10 MPa. AAC is mainly used for non-load-bearing walls or floors and has a great reputation for its excellent insulation properties and low density. Foaming agents are attracting more and more attention to be used for the production of cellular lightweight concrete, which can be cured like normal concrete.

The properties of lightweight aggregate concrete are dependent on the properties of the aggregates to a great extent. Fig.1 shows the effect of aggregate nature on the density of concrete made with a variety of lightweight aggregates. The concretes can be classified into three catalogues [1]: low-density concretes, moderate strength concretes and structural lightweight concretes. Low-density concretes are very light are employed chiefly for nonstructural insulation purposes. They have low unit weights, seldom
exceeding 800 kg/m$^3$, low thermal conductivity and low compressive strengths, usually less than 7 MPa. Structural concretes use aggregates that fall on the other end of the scale and that are generally made with expanded shales, clays, slates, slags, pumice and scoria. They have a minimal strength of 17 MPa and can reach greater than 35 MPa with most of these aggregates. Since the unit weights of structural lightweight aggregate concretes are considerably greater than those of low-density concretes, insulation efficiency is lower. However, thermal conductivity values for structural lightweight concrete are substantially better than for normal weight concrete. Moderate strength concretes fall about midway between the structural and low-density concretes. Their compressive strengths range 7 to 25 MPa and insulation characteristics are intermediate.

Fibers are widely used in concrete for a number of applications. Very limited researches have been conducted on fiber-reinforced lightweight concrete. It is indicated that the use of fiber can also increase the modulus of elasticity and ductility of lightweight concrete very significantly [3]. In this paper, lightweight aggregate concrete with strengths from 8 to 50 MPa and dry densities from 800 to 1400 kg/m$^3$ were developed for both non-load bearing and load bearing applications. Fibers were added to improve the ductility and handling properties of the concrete.

**RESEARCH SIGNIFICANCE**

Lightweight concrete is being used more and more for structural and non-structural applications. Fibers are widely used in concrete for improvement of ductility of concrete However, very limited researches have been conducted on fiber-reinforced lightweight concrete. This paper presents some properties of fiber-reinforced concrete with strengths from 8 to 50 MPa and dry densities 800 to 1400 kg/m$^3$. The results will provide technical information for the design and application of lightweight concrete for both structural and non-structural uses.

**EXPERIMENTATION**

**Materials**

The cementing materials used in this study include ASTM Type I and III portland cement, ASTM Grade 120 slag cement, and ASTM Class F coal fly ash. The replacement of cement with the slag or fly ash varied from 20 to 50%. Expanded shales and clays, which meets the requirements of ASTM C 330, were used as lightweight fine and coarse aggregates. Foaming agents and aluminum powders were used to introduce air bubbles in cement paste in the concrete. Chopped polypropylene and nylon fibers with length from 10 mm were used. Their dosage was 6 kg/m$^3$ of concrete.

**Mixing and Preparation of Specimens**

A wide range of mixing proportions were designed to produce concrete with different strengths and densities. The concrete mixtures were mixed, placed and finished like conventional concrete. Cylinders of 10x20 cm or cubes of 10x10x10 cm were
prepared for compressive strength testing, 10x10x35 cm beams were used for flexural strength testing and 7.5x7.5x28 cm prisms were used for drying shrinkage measurement. Some specimens fire testing have a size of 15x50x2 cm and a size of 30x30x5 cm for thermal conductivity measurement.

Two curing scenarios were used for those specimens for strength testing: standard room temperature curing and steam curing. For standard room temperature curing, 24 hours after the casting, specimens were demolded and then cured in lime-saturated water at 23±3°C until testing ages. Presented result is an average of three testing specimens. For steam curing, the specimens were cured with molds. The curing scenario was 5 hours of preset time, 3 hours of temperature raise time, 8 hours of constant temperature of 75°C and 3 hours of cooling time. All other specimens were cured at room temperatures.

Strength Testing

If the specimens were cured in steam, they were demoulded and tested once the specimens are cooled down to room temperature. If they are cured at room temperature, they were demoulded in the second day and tested for strength at 1, 3, 7 and 28 days. Flexural strength was measured using the third-point loading test following ASTM C78.

Drying Shrinkage and Mass Loss Measurements

The specimens were tested for drying shrinkage and mass loss at relative humidity of 50±5% and 23±2°C. As for the steam-cured specimens, they were sealed to cool down and then taken initial measurements of length and mass in the second day after the steam curing. As for room temperature curing, specimens were in a fog room until 28 days for initial measurements of length and mass.

Fire Testing

In this work, a welding torch fire was used to heat the specimens. The behavior of the specimen during the fire testing was visually observed and photographed.

RESULTS AND DISCUSSION

Compressive Strength

For given materials, the strength of the fiber-reinforced lightweight concrete is dependent upon the density of the concrete. Fig.2 shows the relationship between compressive strength and dry density of concrete made with from 0 to 50% of ASTM F fly ash and Grade 120 slag cement after steam curing. It is noticed that the strength of these concretes continued to increase with time when concrete samples were kept in moist conditions because fly ash and slag hydrate more slowly than Portland cement.

The addition of fiber has no or little effects on compressive strength. However, it was found that the use of fiber significantly increases the impact resistance and ductility.
of concrete. After compression testing, it was noticed that the conventional concrete specimen cracked along 45° diagonal line and broke, while the lightweight concrete specimen cracked but still remained in whole piece.

**Stress-Strain Relationship and Modulus of Elasticity**

Fig.3 is a typical stress-strain relationship of the lightweight concrete with a compressive strength approximately 27 MPa. For normal weight concretes, they reach their ultimate compressive strength at strains of 0.002 regardless of strengths [4]. The results in Fig. 3 indicated that the lightweight concrete fails at strain of around 0.003.

The secant modulus of elasticity of the materials calculated based on the strain at about 40% of the ultimate strength is around 10.7 GPa, which is about half of regular concrete. ACI 318 code states that the modulus of concrete (Ec) with a unit mass from 1400 to 2500 kg/m$^3$ can be calculated as follows [5]:

$$E_c = 0.043W_c^{1.5} \sqrt{f'_c}$$

where:

- $W_c$ = unit mass of concrete, kg/m$^3$;
- $f'_c$ = compressive strength of concrete at 28 days, MPa.

Based on this equation, the calculated modulus of modulus elasticity of the lightweight concrete was 11.6 GPa. Thus, the measured and calculated Modulus of elasticity of lightweight concrete are very close and the ACI Equation is valid for the estimation of modulus elasticity of the lightweight concrete based on its unit weight and compressive strength.

**Flexural Strength**

During flexural testing, it was noticed that the specimen did not break into two pieces even with a broad crack developed due to the presence of fiber. The flexural load and displacement relationship is shown in Fig.4. It can be seen that a very high residual strength can be still measured after the cracking. It was noticed that the residual strength depended on the strength of the concrete, and the nature and dosage of the fiber used in the concrete. As described in previous publications, the introduction of fiber did not show a noticeable effect on flexural strength of the lightweight concrete.

It was observed that, within the studied strength range, the compressive/flexural strength ratio increased linearly with compressive strength of concrete as shown in Fig 5. When the compressive strength is about 6 MPa, the compressive/flexural strength ratio was around 3. As the compressive strength reaches 45 MPa, the compressive/flexural strength ratio reaches approximately 10, which is a typical value for conventional concrete.
Drying Shrinkage

Generally speaking, the shrinkage of the concrete increased with the decrease of strength. The drying shrinkage of batch with a 28-day strength of about 30 MPa, cured in steam at 75°C and in fog room at 23°C, is shown in Fig. 6. Results from a conventional concrete with a water to cement ratio of 0.4 and 28-day strength of 45 MPa, are also plotted for reference. It can be seen that the lightweight concrete displayed much lower shrinkage than the conventional concrete during initial testing period. Although the gap decreased with testing time, the conventional concrete still showed higher shrinkage than the lightweight concrete even after 6 months of testing time. Initially, steam-cured specimens showed higher shrinkage than the specimens cured at room temperature. However, the former gave lower shrinkage after about 7 months of testing.

Fig. 7 is the mass loss during the shrinkage measurements. Although the moisture loss of the conventional concrete is much lower than the lightweight concrete, the former showed much higher shrinkage than the latter as described above. The drying shrinkage behavior of concrete materials can be described by the pore size distribution and thermodynamic behavior of water in the pores. Pore size distribution can be characterized by $r_s$, which is defined as the radius of the pores where the meniscus forms; i.e., the pores whose radii are smaller than $r_s$ are assumed to be filled with liquid water while pores larger than this are dry. As the drying progresses, the parameter $r_s$ would decrease. The smaller, the larger the capillary tensile forces set up at the meniscus, hence higher the resulting shrinkage. In the lightweight, the moisture comes from large air voids, while the moisture evaporated from regular concrete is mainly from small pores. This explains the difference between the two types of concrete.

Thermal Conductivity

The density of concrete determines its thermal conductivity. Fig. 8 is a relationship between density and thermal conductivity of concrete developed by Valore [6]. Testing of concrete made with different materials such as low carbon fly ash, high carbon fly ash, blast furnace and ground glass has indicated that the thermal conductivity-density relationship for fiber-reinforced non-autoclaved cellular lightweight concrete still follows the relationship as illustrated in Figure 8, and the use of those different materials does not have an obvious effect on the thermal conductivity of concrete for a given density.

Fire Resistance

Fig. 9 shows the torch-fire burning of a small lightweight concrete specimen. A small piece of copper was placed in the heating spot and molten. However, no flame or spalling was observed during the burning process. Fig. 10 is a picture of the specimen after the torch-fire burning.
CONCLUSIONS

There is a good relationship between the density and compressive strength of the concrete. The compressive/flexural strength ratio decreased linearly as the compressive strength increase.

The introduction of fiber does not show an obvious effect the ultimate strength of the concrete, but it provide a residual strength, which is dependent on the strength of the concrete, and the nature and dosage of the fiber. The presence of fiber improves the handling properties of hardened concrete significantly, especially in the low strength range.

The lightweight concrete has a higher moisture loss than but a lower shrinkage than the conventional concrete. The density of the concrete determines its thermal conductivity regardless of the raw materials used.

The lightweight concrete showed good fire resistance and did not generate spalling during the torch fire burning.

REFERENCES

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5. ACI 318, Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, 2002.
Figure 1: Approximate Unit Weight and Classification of Lightweight Aggregate Concretes [1].

Figure 2: Relationship Between Strength and Bulk Density of Fiber-Reinforced Lightweight Concrete.

Figure 3: Compressive Stress-Strain Relationship of Lightweight Concrete.
Figure 4: Flexural Load-Deflection Relationship of Fiber-Reinforced Cellular Lightweight Concrete.

Figure 5: Relationship Between Compressive Strength and Compressive/Flexural Strength Ratio.

Figure 6: Drying Shrinkage of Cellular Lightweight and Conventional Concretes.
Figure 7: Moisture Loss of Cellular Lightweight and Conventional Concretes During Drying.

Figure 8: Relationships Between Density and Thermal Conductivity of Concrete [6].

Figure 9: Torch Fire Burning of Specimen.
Figure 10: Specimen after Torch-Fire Burning.
Acoustically Efficient Concretes Through Engineered Pore Structure

by N. Neithalath, J. Weiss, and J. Olek

**Synopsis:** Three classes of specialty cementitious materials were evaluated for their potential benefits in sound absorption including a Foamed Cellular Concrete (FCC) with density ranging from 400 – 700 kg/m$^3$, Enhanced Porosity Concrete (EPC) incorporating 20-25% open porosity, and a Cellulose Cement Composite (CCC) with density 1400 – 1700 kg/m$^3$. Cylindrical specimens of these materials were tested for acoustic absorption in an impedance tube. The FCC specimens showed absorption coefficients ranging from 0.20 to 0.30, the higher value for lower density specimens. The closed disconnected pore network of FCC hinders sound propagation, thereby resulting in a reduced absorption, even though the porosity is relatively high. The most beneficial acoustic absorption was observed for EPC mixtures. When gap-graded with proper aggregate sizes, these no-fines EPC mixtures dissipate sound energy inside the material through frictional losses. The cellulose fiber cement composites use cellulose fibers at high volume fractions (~7.5%), which are believed to provide continuous channels inside the material where the sound energy can be attenuated. By engineering the pore structure (by careful aggregate grading as in EPC, or incorporating porous inclusions like morphologically altered cellulose fibers) cementitious materials that have the potential for significant acoustic absorption could be developed.

**Keywords:** acoustic absorption; cellulose cement composite; damping; enhanced porosity concrete; foamed cellular concrete; impedance; porosity
INTRODUCTION

Noise pollution affects more people than any other kind of pollution in the modern industrialized world (1). In the United States, more people are exposed to highway noise than from any other single noise source (2). Noise pollution is especially annoying in densely congested urban settings where residents live near highways and main transportation thoroughfares. The need to control noise in such environments therefore offers an incentive to study the acoustic performance of cementitious materials. Conventional concrete is a preferred material for the construction of noise barriers due to its excellent performance as a sound reflecting material, but its sound absorbing capability is extremely limited. While the construction of sound barriers impedes the sound transmission path between vehicles and the residential and commercial development located alongside the highways resulting in noise abatement, they tend to be extremely costly, unsightly, and not practical for bridges and/or urban highways (1, 3). A better method to control noise is the use of sound absorbing materials, or pavement surfaces that result in less noise generation. It is common practice in noise control engineering to use reduced density porous materials to achieve sound absorption. Sound absorbing materials are used for walls, floors, and ceiling to reduce the noise that is generated from within an enclosure.

This paper describes the results of a research study aimed at developing and characterizing concretes with specially engineered pore structure as materials for sound absorption. In general, sound absorbing materials are porous materials with reduced densities. The basic principle is that the acoustic energy is converted into heat energy in the open pores of the material, resulting in the reduction of perceived noise levels.
CONCEPT OF SOUND ABSORBING MATERIALS

Cementitious materials can be classified as “rigid-framed” (the rigidity or stiffness of the frame is much higher than that of air) materials where sound absorption is typically believed to occur in an array of tortuous pores in the material (4,5). This acoustic loss is attributed to the adiabatic pressure changes due to air compression (as the air enters the pores) and expansion (as air leaves out of the pores). In addition, the frictional losses in the pore walls also contribute to the acoustic absorption.

MATERIALS AND MIXTURES CONSIDERED

Three different cementitious materials with different porosity were considered for this study. The method of manufacture and properties of these materials are described in this section.

Foamed Cellular Concrete (FCC)

Foamed cellular concrete is manufactured by mixing cement, sand, and a foaming agent. The material used in the study was a commercial product, and was precast in the plant. The density of the material can be varied from 300 to 1200 kg/m$^3$. Three different densities, as provided by the manufacturer, were used in this study – 450, 560, and 700 kg/m$^3$. Figure 1(a) shows a cross section of foamed cellular concrete.

Enhanced Porosity Concrete (EPC)

Enhanced porosity concrete (EPC) is proportioned by gap grading the coarse aggregates and either eliminating (or limiting) the sand volume in the matrix. Figure 1(b) shows a typical cross section of EPC. The cement content is established by providing a sufficient amount of paste to coat the aggregates since an excessive amount of paste may drain through the pores of the material. The water-to-cement ratio is also kept low for the same reason.

Three aggregate sizes - #8, #4, and 3/8” were chosen for single sized aggregate mixtures. The aggregate sizes shown refer to the sieve in which they were retained (for instance, # 4 indicates that the aggregates passed through a 3/8” (9.5 mm) sieve and were retained on the # 4 (4.75 mm) sieve). In addition, binary blends of these mixtures were also considered. Blends were prepared by replacing 25, 50, and 75% by weight of the larger sized aggregates successively by smaller sized aggregates. The mixtures were prepared using a laboratory mixer in accordance with ASTM C 192-00 (6), cast in 150 x 150 x 700 mm beam molds and consolidated using external vibration. Cylindrical specimens, 95 mm in diameter were cored from these beams at a later age for acoustic absorption measurements in the impedance tube.
Cellulose Cement Composite (CCC)

Morphologically altered cellulose fibers were considered as an option to introduce sound absorbing channels in the material. The cellulose fibers used in this study were in the form of nodules, 2-8 mm in size, and are referred to as "macronodules" (Figure 1(c)). These nodules are aggregations of individual fibers and therefore are porous. A cement-sand mortar with 50% aggregate volume was used. The fiber contents used were 1.5, 3.0, 4.5, 6.0 and 7.5% by total volume.

Cement and sand were first mixed at low speed for one minute and then the fibers were added, during mixing. Approximately three quarters of the water was needed for proper mixing. The water was added and all ingredients were mixed at medium speed for two minutes. The remaining water was then added with water reducer and mixed until a uniform mixture was obtained (typically one minute). Care was taken to ensure that the mixer did not run at a higher than required speed or for a longer than required duration to avoid breaking down of fiber nodules in the mixer. For mixtures with high volumes of fiber (6.0 and 7.5%), an accelerator was added since it was noticed that there was considerable set retardation otherwise. Cylindrical specimens (95 mm in diameter) were prepared for acoustic absorption whereas beams (75 mm x 250 mm x 25 mm) were made for measurements of damping behavior.

EXPERIMENTAL PROCEDURES

Porosity Determination

Foamed Cellular Concrete (FCC) – The porosities of the FCC specimens could be related to their densities. FCC with a density of 450 kg/m$^3$ had a porosity of 0.75, the one with a density of 560 kg/m$^3$ had a porosity of 0.70 and the one with density 700 kg/m$^3$, 0.61.

Enhanced Porosity Concrete (EPC) – Because of the presence of large interconnected pores in the EPC system, the following procedure was developed to determine the porosity. The cylindrical specimens (95 mm in diameter and 150 mm long) that were cored from beams were immersed in water for 24 hours to saturate the pores in the matrix. After this period, the sample was removed from water, and the excess water on the sides wiped to bring it to SSD condition. The sample was then enclosed in a latex membrane and the bottom of the cylinder was sealed to a stainless steel plate using silicone sealant. The mass of the sample, latex membrane, and the steel plate together ($M_1$) was measured. Water was added to the top of the sample until it was filled, which indicated that all the interconnected pores were saturated. The mass of the system filled with water was taken ($M_2$). The difference in the mass $\Delta M = (M_2 - M_1)$ represents the water in the pores. This mass was converted into an equivalent volume of water, and expressed as a percentage of the total volume of the specimen to provide an indication of the total porosity.

Cellulose Cement Composite (CCC) – Porosity was determined on 75 mm x 75 mm x 25 mm prisms of composite specimens. Vacuum saturation (as described in RILEM CPC 11.3 (7)) has been followed to determine the porosity. The prisms were dried in an oven
at 10565°C until no change in measured weight was noticed. The specimens were then placed in a vacuum chamber for 3 hours before water was introduced to the chamber, under vacuum. The vacuum was maintained for 6 more hours after which time the specimens were left in water for additional 18 hours. The saturated surface dried weight was then determined. The water absorbed by the fibers was accounted for in the vacuum saturated weight so as to obtain the total porosity.

Determination of Acoustic Absorption Coefficient ($\alpha$)

The acoustic absorption coefficient ($\alpha$) is a measure of how well a material can absorb sound. When a sound wave strikes a material, a portion of the sound energy is reflected back while a portion is absorbed by the material. The absorption coefficient is the ratio of the absorbed energy to the total incident energy.

The acoustic impedance tube was used to determine the absorption coefficient ($\alpha$) using the experimental set up as described in ASTM E 1050-98 (8) (Figure 2). The sample was placed at one end of the cylindrical tube with a rigid backing. The specimen was tested with a plane acoustic wave propagating along the axis of the tube. The absorption coefficient is calculated as:

$$\alpha = 1 - |R|^2$$  \hspace{1cm} (Equation 1)

where the reflection coefficient (R) is computed for frequencies ranging from 100 to 1600 Hz using the following equation:

$$R = \frac{e^{jkd_1} - e^{jkd_2} \cdot P}{e^{-jkd_2} \cdot P - e^{-jkd_1}}$$  \hspace{1cm} (Equation 2)

where $d_1$ and $d_2$ are the distances from the specimen surface to the first and second microphones respectively (Figure 2), $j$ is $\sqrt{-1}$, $k$ is the wave number (ratio of angular frequency to the wave speed in the medium), and $P$ is the ratio of acoustic pressures.

A threshold of 100 Hz was established because at very low frequencies, the acoustic pressures were difficult to stabilize. Frequencies higher than 1600 Hz can be measured accurately only when the impedance tube has a small diameter (9). To achieve acoustic measurements over the widest range of frequencies, and to ensure that a “standing wave” is generated inside the impedance tube, the diameter of the cylinder should be as small as possible. However, preparation of homogeneous concrete samples of such small sizes tends to be difficult due to the size of the aggregates. An upper limit of 1600 Hz was used because the most prominent levels in tire-pavement interaction noise occurs between 800 and 1200 Hz (1, 10).

Specific Damping Capacity for Cellulose Cement Composites

The Specific Damping Capacity ($\chi$) was determined on 75 mm x 250 mm x 25 mm beams according to the decaying sin wave method.

$$\chi = \frac{A_i - A_{n+i}}{A_i} \times 100\%$$  \hspace{1cm} (Equation 3)
where $A_i$ is the amplitude of the $i^{th}$ period and $A_{n+i}$, that of $(n+i)^{th}$ period.

RESULTS AND DISCUSSIONS

This section presents the results of the experimental investigations conducted on foamed cellular concrete (FCC), enhanced porosity concrete (ECC), and cellulose cement composites (CCC) to ascertain their acoustical efficiency.

Foamed Cellular Concrete (FCC)

The acoustic absorption spectra (variation of acoustic absorption coefficient with frequency) of FCC are given in Figure 3(a). It can readily be noticed that the peak absorption coefficient is reduced with increasing specimen density. However, the maximum absorption coefficient ($\alpha$) of FCC is higher than that of normal mortar or concrete. For the chosen specimen length (150 mm), the peak absorption coefficients occur at a frequency of 300 – 400 Hz. The slight variations in the frequency at the peak absorption are due to the fact that there are changes in the pore structure of the material with changes in density. Figure 3(b) shows the maximum absorption coefficients of all the three FCC samples plotted against density. The maximum absorption coefficients are in the range of 0.20-0.30. Though the FCC specimens are very light and have a high porosity, the absorption coefficients are not particularly high due to their disconnected pore structure, even though they are superior to normal concrete or mortar, which has an $\alpha$ of about 0.05.

Enhanced Porosity Concrete (EPC)

The porosities of EPC (proportioned with single sized aggregates) are shown in Figure 4(a) and the acoustic absorption spectra of these mixtures in Figure 4(b). Though the porosities of these mixtures lie in a very narrow range (0.19-0.21), the mixture with larger aggregate size (3/8”) tend to be acoustically less efficient because of the large pore sizes in these materials (characteristic pore size, which is the median of all pore sizes greater than 1 mm in the material, is ~ 5mm for mixtures with 3/8” aggregates, as compared to ~3 mm in mixtures with smaller sized aggregates) which do not force the sound waves to alternatively compress and expand, which is the primary energy expending process in these materials (10). This points to the fact that though the total pore volume is a very important pore structure feature as far as acoustic absorption is concerned, the pore sizes also play a significant role. Large pore sizes are found to be acoustically inefficient. The acoustic performance of mixtures comprising of either #4 or #8 aggregates alone was comparable.

As described earlier, blends of two different aggregate sizes were also used to evaluate their efficiency in acoustic absorption. Blending of aggregates of different sizes is expected to generate optimal porosity and pore size in EPC thus aiding in acoustic absorption. A typical case of blending of #4 and #8 aggregates is described in this section. The porosities of the blended mixtures are shown in Figure 5(a). It can be seen that the highest porosity is achieved for a 50% #4, 50% #8 blend. This can be explained
by the fact that there is an increased volume of voids in the interface in a mixture of coarse and fine particles (11). The characteristic pore size of the EPC mixture with #4 aggregate was found to be smaller than 2.36 mm, which is the size of #8 aggregate. Therefore the smaller aggregate could not fit into the pore space of the mixture with larger size aggregates, resulting in a higher porosity. Similar explanation also applies for the 75% #4, 25% #8 blend.

The acoustic absorption spectra for 150 mm long EPC mixtures with a blend of #4 and #8 aggregates are given in Figure 5(b). It can be seen that for the 75% #4, 25% #8 and 50% #4, 50% #8 blends, the maximum acoustic absorption coefficients are higher than those for single sized aggregate mixtures. It should be noted that these are the mixtures with higher porosity, which partly explains the reason for higher acoustic absorption. But more importantly, in these mixtures, the blending of the aggregates creates a more acoustically efficient pore structure (with respect to pore size and tortuosity) leading to better acoustic absorption. A thorough treatment of the acoustic absorption characteristics of EPC with different aggregate sizes and blends could be found elsewhere (10).

Cellulose Cement Composite (CCC)

The use of morphologically altered cellulose fibers for acoustic effectiveness was based on the premise that these fibers, because of their physical nature, could provide continuous pathways inside the material through which sound waves can propagate, and attenuate (12). Originally, three different morphologies of cellulose fibers were considered, but only the macronodule fiber, which was the most acoustically effective, is discussed in this paper.

The porosity of the composite was found to increase with an increase in volume of the fiber phase, as shown in Figure 6(a). The relationship between the porosity of the composite (\(\phi_{\text{composite}}\)) at any fiber volume and the porosity of the fiber free mortar matrix (\(\phi_{\text{mortar}}\)) can be given by:

\[
\phi_{\text{composite}} = \phi_{\text{mortar}} (1 + AV_f) \tag{Equation 4}
\]

where the value of the constant A can be considered as an indicator of the contribution of the fiber phase to the total porosity of the composite.

The acoustic absorption spectra for CCC with macronodule fibers are shown in Figure 6(b). For these 75 mm long specimens, the absorption peak occurs at a frequency of approximately 500 Hz. It can be seen that an increase in fiber content increases the maximum absorption coefficient. For a sample with no fibers, the maximum absorption coefficient (\(\alpha\)) is approximately 0.05 and it steadily increases to approximately 0.40 for the composite with 7.5% volume of macro nodules. The macro nodules appear to provide porous channels inside the specimen where the incident sound energy can enter and attenuate. With an increase in fiber volume, it is expected that there is an increase in the number of connected porous channels, leading to an increase in sound absorption.

The energy dissipation capacity of a material can also be defined in terms of its specific damping capacity. This parameter is very useful in characterizing materials like...
CCC where an inclusion phase is present, the stiffness of which differs from that of the matrix by more than one order of magnitude (the modulus of elasticity of normal mortar is approximately 30 GPa, where as the cellulose fibers have a modulus of 1-2 GPa). The acoustical mismatch of impedance at the interface of the constituent phases contributes significantly to damping (12, 13).

For specimens with macronodules, Figure 6(c) shows the relationship between fiber content and specific damping capacity for two different ages of curing and three different moisture conditions (wet, dry, and rewetted). An increase in fiber volume results in an increased damping capacity, especially for wet specimens. This may be attributed to the fact that an increase in volume of macro nodules increases the stiffness mismatch, resulting in higher energy dissipation in the material than it would have for a sample without fibers. These results are also in line with observations from a study on damping mechanisms in hardened pastes, mortar and concrete which indicated that the damping capacity is related to the percentage of water-filled pores in the system (14), with increased moisture leading to a higher degree of damping. Higher volumes of macro nodules effectively increase the amount of water filled pores in the system, thereby resulting in high damping capacity values. For the same curing conditions (7 day and 14 day wet), it can be observed that the damping capacity decreases with age, probably due to reduction in porosity and pore water content as a result of cement hydration. The reduction, though, is not very large in this case.

The damping capacity is extremely sensitive to the moisture content. The values were reduced to one-fifth of the measured saturated values for composites reinforced with 7.5% macro nodules when the specimens were dried at 105°C. The loss of moisture and development of microcracking may have opposing effects on damping (14). The presence of microcracks increases damping whereas the loss of moisture decreases damping. When the specimens are dried at 105°C, there are chances of formation of microcracks, but it appears that the increase in damping capacity due to microcracking is much smaller than the decrease due to water loss. As a result, dry specimens possess a smaller damping capacity than wet ones. The variation in damping capacity with fiber volume is also smaller for dried specimens. This brings out another interesting observation. Though the acoustical mismatch between the different phases in the material may seem to be the driving force for increased damping of composites with higher fiber volumes, the influence of presence of large amounts of water in these mixes cannot be neglected. On rewetting of the 14 day old specimens, it can be seen that, for composites with macro nodules, the damping capacity increases again. The increase this time is very significant and the value is higher than that observed for 7 day moist cured mixes, especially at low fiber contents. This could be due to the synergestic effects of both microcracking as well as the presence of water molecules. At higher fiber contents, this value approaches the damping capacity observed for 7 day and 14 day wet composites (12).
SUMMARY AND CONCLUSIONS

Three porous materials, having different pore volume fractions, and vastly varying pore structure characteristics have been studied for their effectiveness in acoustic absorption. It has been found that the pore structure could be tailored to achieve desirable acoustic absorption characteristics. In the case of EPC, this could be accomplished by blending different aggregate sizes in chosen proportions, whereas for CCC, the use of morphologically altered fibers to provide continuous channels in the material is an option. The closed cell structure of FCC is not as effective as the other two materials in acoustic absorption.

The conclusions from this study can be summarized as follows:

(i) FCC though having a higher porosity, has a maximum acoustic absorption coefficient ($\alpha$) in the range of only 0.20-0.30 because of its closed cell structure. However, this value is higher than that of normal concrete (~ 0.05). CCC incorporating high fiber volumes show $\alpha$ values of about 0.40, where as the $\alpha$ values of EPC were observed to be as high as 0.80.

(ii) Acoustic absorption coefficient of EPC with larger sized aggregates is found to be typically lower, since the pore sizes also tend to be large. Larger pore sizes are acoustically inefficient.

(iii) Blending of selected aggregate sizes in chosen proportions is found to create pore sizes in EPC that are acoustically efficient. Higher porosity of the blends, along with a pore structure that is tortuous enough to absorb sound waves, is believed to be the reason for the improved acoustic absorption. The acoustic absorption of EPC with properly chosen aggregate blends is around 0.80.

(iv) The use of morphologically altered cellulose fibers results in a moderate acoustic absorption. The porous macronodules are expected to provide interconnected channels inside the material where the sound waves can attenuate. The absorption coefficient increases with an increase in fiber volume, possibly due to increased porosity, and the generation of increased number of interconnected porous channels in the matrix.

(v) Specific damping capacity increases with an increase in fiber content, presumably due to an increased impedance mismatch between the cementitious matrix and the cellulose phases.

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Figure 1: (a) Typical cross section of FCC, (b) Typical cross section of EPC, (c) Macronodule fibers used to manufacture CCC

Figure 2: Impedance tube set up to measure acoustic absorption

Figure 3: (a) Acoustic absorption spectra of FCC, (b) Variation of maximum absorption coefficient of FCC with density
Figure 4: (a) Porosity of EPC with single sized aggregates (b) Acoustic absorption spectra of EPC with single sized aggregates. The values were obtained for each 1 Hz frequency, but the symbols are shown discretely to distinguish between different specimens.

Figure 5: (a) Porosity of EPC with blended aggregate mixtures (b) Acoustic absorption spectra of EPC with a blend of #4 and #8 aggregates.
Figure 6: (a) Influence of fiber volume on the porosity of CCC (b) Acoustic absorption spectra of CCC with varying fiber volume (c) Relationship between fiber volume and specific damping capacity of CCC