Evaluation of Existing Structural Design Methods for AAC Panels

Designing with Autoclaved Aerated Concrete

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REPORT SUMMARY

Autoclaved Aerated Concrete (AAC) is a lightweight concrete with no coarse aggregate. It is produced by mixing portland cement, lime, aluminum powder, and water with a large proportion of a silica-rich material. In many of the forty countries with existing AAC industries, sand is the source of that silica. But fly ash has been used successfully in England and China for over 25 years. Fly ash can be as much as 75% of the material by weight. In England nearly 1 million tons of fly ash is used annually to produce AAC concrete.

Background

AAC concrete was first introduced in Sweden in the late 1920s. The growth rate of AAC materials has been spectacular. In 1991, 8.65 million cubic meters of AAC were produced in 67 factories in Western Europe and 10 to 12 million cubic meters produced in 120 factories in Eastern Europe. In addition there are over 70 plants in Asia. The first AAC plant in the U.S. was opened in 1996 in Georgia. Since then others have been opened in Florida and Tennessee. One difficulty in introducing a completely "new" material into the building industry in the U.S. is that there is a lack of applicable building codes and design methodology for working with a new type of building material.

Objectives

To improve the understanding of AAC materials by an evaluation of existing international structural design methods for AAC panels. To recommend a design approach that would be suitable for the American engineering design community.

Approach

The EPRI researchers conducted a detailed review of existing literature on design methodologies for reinforced AAC panels. Also, design methods were evaluated and laboratory analytical studies were conducted to compare results obtained among different methods and existing test data.

The evaluation of design methods focused on strength provisions such as flexure, shear, axial load, and anchorage of reinforcement; as well as serviceability conditions such as deflection limits, deflection computations, crack limits, and bearing at supports. The evaluation was conducted by means of sample computations. The conclusions and recommendations are based on the comparison among methods and their differences with respect to existing test data.

Results

Based on the review of different methodologies, it is clear that an American AAC methodology should address at least the following design topics: (1) Bending strength of members with single or multiple layers of reinforcement, (2) Shear strength of bending members with or without shear reinforcement, (3) Axial strength of members with or without eccentricity and including considerations for buckling and second order effects, (4) Shear strength of axially loaded members, (5) Anchorage of reinforcement, (6) Bonding, (7) Bearing strength, and (8) Punching strength. Also the following serviceability conditions should be addressed: (1) Short and long term deflections, (2) Deflection limits, (3) Cracking load and cracking moment, and (4) Crack control. The literature review found little information specifically related to the design of fly ash based AAC, because code agencies had decided that the source of silica in the material, sand or fly ash, was immaterial.

The development of a completely new U.S. based design methodology for reinforced AAC that covers the design aspects previously enumerated may require extensive testing and calibration and important resources in terms of money and time. To address those issues, a two-step research program based on the current state of knowledge is recommended for the development of a U.S. methodology for the design of reinforced AAC panels.

EPRI Perspective

As part of EPRI's effort to find additional uses for flyash, it was sought to develop market acceptance for AAC concrete in North America. A first step to design and fabricate a mobile demonstration plant that toured ten coal burning power plants at utility sites during 1993-1995. During a six-week stay at each plant, it produced about 1,500 blocks for demonstration purposes by each utility. Since AAC concrete was a new material to the US marketplace, this earlier EPRI project enabled members of the local construction community to witness the production process first-hand and to see the blocks used in some field demonstration. Another aspect of the previous EPRI sponsored work to develop this technology was an engineering and environmental evaluation of the AAC materials, which was completed in 1996 (TR-105821, V1-V3).

The AAC industry in the U.S. is coming of age. During 2000, the industry trade organization, the Autoclaved Aerated Concrete Products Association, was formed. This group will be responsible for promoting this new material in the North American marketplace and will be encouraged to develop design guidelines according to the principles documented in this report.

ABSTRACT

The present study is based on a review and evaluation of seven design methodologies found through a literature search. The methodologies have been compared to design values obtained from test data. Based on the review, evaluation, and comparison of the design methodologies, a two-step approach is presented for the development of a U.S. methodology for the design of reinforced Autoclaved Aerated Concrete (AAC) panels.

Reinforced AAC products are used as roof and floor slabs, wall panels that are loadbearing or non-loadbearing, as well as lintels. The success of AAC products around the world is a favorable factor in predicting success for this product in North America. It is successfully competing against a wide-variety of traditional building materials in over 40 countries in every climatic region of the globe. It has established itself as a universally applicable building material. The numerous advantages in it's characteristics, the fact that it is a completely anorganic composition, and the fact that it provides an alternative to land disposal for flyash makes AAC concrete a very attractive choice. This light-weighted porous, chemically resistant, easily workable material is more and more a product with a future, as the emphasis on specific properties like thermal insulation is growing.

Phase I of the work consisted of selecting the most appropriate of the existing foreign methods and adapting these methods according to U.S. design practice. The evaluation of design methods focused on strength provisions such as flexure, shear, axial load, and anchorage of reinforcement; as well as serviceability conditions such as deflection limits, deflection computations, crack limits, and bearing at supports. The evaluation was conducted by means of sample computations. The conclusions and recommendations are based on the comparison among methods and their differences with respect to existing test data. In Phase II, an expanded program of needed research is outlined to confirm the proposed design methods and procedures with emphasis on fly ash based AAC.

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1

INTRODUCTION

Autoclaved aerated concrete has been used in construction for over 70 years after its original development in Sweden in 1929. AAC is a lightweight artificial stone of uniform cellular structure with exceptional properties. The raw materials for the manufacture of AAC consist of cement, sand or fly ash, and lime. Fly ash based AAC, although tested by decades of manufacture and use in other countries, only recently entered the United States market. Fly ash AAC possesses the same high qualities as sand AAC, but with the added benefit of fly ash recycling and a lowered energy requirement for production.

As stated, AAC was developed in Europe, thus producing vast amounts of foreign literature. Because AAC is a relatively new type of building material in the United States, several barriers do exist that render the large body of information available in foreign literature not directly applicable in the U.S. The existing information is not available in a well-defined, unified form that can easily be adopted into U.S. standards and codes. Further, the foreign standards and codes vary considerably from one country to another in their treatment of AAC, and well-documented experimental data to verify the existing theory is seldom available.

Clearly, a detailed, uniform design method on AAC panels is needed to enable designers to confidently utilize and specify AAC for various construction projects in the U.S. Therefore, all foreign design methods are needed for research and evaluation in order to develop a common standard in the U.S. The success of this objective is necessary for the establishment of AAC as a reliable building material in the U.S.

1.1 Research Objectives

The main objective of this study is to provide a detailed evaluation and critique of existing foreign design methods and procedures for sand and fly ash based AAC panels. The existing methods are assessed and recommendations are made based on comparisons among methods and existing test data. Another objective is to establish whether the methods studied are adequate for incorporation into U.S. practice. Finally, this study is intended to provide recommendations about the requirements to develop a U.S. design methodology for reinforced AAC members. This U.S. design methodology may be based on existing methodologies or a completely new method developed through experimentation and testing.

1.2 Literature Review

The literature review includes a comprehensive study of all the available design methods on reinforced AAC panels. The review focuses on domestic and foreign literature. Extensive library and Internet searches were conducted to compile a list of references related to design methodologies for AAC. The complete list of references is presented in Chapter 9.

Much of the literature found on AAC contains large amounts of information on its material properties and the processes involved in producing the product, but little on design of AAC components. One of these references includes a book entitled "Autoclaved Aerated Concrete, Moisture and Properties" by Folker H. Wittman (69), in which only two articles by Dietmar Briesman (11) and Cividini (17) address issues of design of AAC.

A major focus of the literature review was to produce data and design information on fly ash based AAC. Very little research has been done in this area, therefore very few documents were found in the literature. A study addressing fly ash AAC and its engineering properties was sponsored by Electric Power Research Institute (EPRI) and performed at the University of Pittsburgh (64, 65, 66). This study concentrates on the use of fly ash AAC in construction and the importance of utilizing AAC as a means of controlling the environmental problems of coal fly ash. Also, an article by Hu (33) addresses the strength characteristics of AAC with fly ash.

A number of major documents that address the design of AAC panels was found in the literature. Each of these documents is used and will be discussed in detail in this report. Other references containing design information are noted in Chapter 9. References include two articles by Aroni (3,4), which discuss the analysis of the shear strength of AAC panels. These articles contain test data that was the basis for the design methods for shear presented in Rilem Recommended Practice (5), which will be discussed later in this report.

Based on the literature review, it was found that a total of seven documents presented relevant information on the design of reinforced AAC panels. The bulk of this report is based on the evaluation of the design methodologies presented in those seven documents. A listing and brief description of the contents of each of these documents follows:

- <u>Rilem Recommended Practice for Autoclaved Aerated Concrete</u>
  This model code, published in 1993, presents a set of recommendations for the design of reinforced AAC members based on an ultimate strength design approach. The topics addressed include bending, shear, axial capacity, deflections, anchorage, and bearing capacity.

- <u>DIN 4223 "Guidelines for the Calculation of Reinforced Roof and Ceiling Panels from Steam-Cured Aerated Concrete and Foamed Concrete"</u>

DIN 4223, published in 1958, contains the German standards for design of AAC panels. DIN 4223 contains information for the analysis of bending, shear, deflections, anchorage and crack formation, based on an allowable stress design approach.

- Danish standard DS 420,"Code of Practice for the Structural Use of Lightweight Concrete"
  DS 420, published in 1983, contains information for the analysis of shear, axial capacity, crack formation, and bearing based on an ultimate strength design approach.

- Draft European Standard PrEN 12602, "Prefabricated Reinforced Components of Autoclaved Aerated Concrete"
  The European standard PrEN 12602, put forth in 1996, is a draft of a technical standard developed by the European Committee for Standardization. This is a detailed document that covers bending, shear, axial capacity, anchorage, bond strength, and deflections, and follows an ultimate strength design approach. This draft is currently under review by the CEN Technical Committee for adoption as a technical standard.

- Autoclaved Aerated Concrete, CEB Manual of Design and Technology
  The Euro-International Committee for Concrete (CEB) presents the Swedish handbook, published in 1978. CEB relies mainly on manufacturer's specifications, but does analyze bearing area and shear stresses in wall panels. It follows an allowable stress design approach.

- SIPOREX Technical Manual for the design of vertical and horizontal slabs used for cladding
  SIPOREX is a manufacturer's manual, which presents information regarding analysis of vertical and horizontal wall panels.

- CSR HEBEL Australia Design Sheets, Design References and Formulae
  The CSR design sheets, published in 1992, are part of a manufacturer's manual providing design aids and formulas for the computation of stresses on reinforced AAC panels.

1.3 Scope of Work

The scope of this study includes a detailed review of existing literature on design methodologies for reinforced AAC panels. Also, design methods are evaluated and analytical studies are conducted to compare results obtained among different methods and existing test data.

The evaluation of design methods is focused on strength provisions such as flexure, shear, axial load, and anchorage of reinforcement; as well as serviceability conditions such as deflection limits, deflection computations, crack limits, and bearing at supports. The evaluation is conducted by means of sample computations. The conclusions and recommendations are based on the comparison among methods and their differences with respect to existing test data.

The present report is organized as follows: An introduction, along with background information and research objectives, is given in Chapter 1. Chapter 2 contains an extensive discussion of each design method. Chapter 3 gives numerical results of design calculations and test results are provides when available. Chapter 4 contains a comparison and a discussion of the results from Chapter 3. Chapter 5 is a critique of each method and is based on Chapters 2, 3, and 4. Chapters 6 and 7 contain conclusions and recommendations regarding the existing design methods and discuss the possible adoption of these methods into U.S. practice. Finally, Chapter 8 provides suggested future work for the development of a common, uniform AAC design standard in the United States.

2

DISCUSSION OF DESIGN METHODS

2.1 General

This chapter contains a discussion and a summary of the evaluated foreign design methods. As indicated previously, based on the literature review, seven methods with the most relevant information on structural design of AAC panels were selected for this study. These methods may be listed as follows:

- Rilem Recommended Practice for Autoclaved Aerated Concrete, 1993
- DIN 4223 “Guidelines for the Calculation of Reinforced Roof and Ceiling Panels from Steam-Cured Aerated Concrete and Foamed Concrete”, 1958
- Danish standard DS 420,”Code of Practice for the Structural Use of Lightweight Concrete”, 1983
- Draft European Standard PrEN 12602, “Prefabricated Reinforced Components of Autoclaved Aerated Concrete”, 1996
- Autoclaved Aerated Concrete, CEB Manual of Design and Technology, 1978
- SIPOREX Technical Manual for the design of vertical and horizontal slabs used for cladding
- CSR HEBEL Australia Design Sheets, Design References and Formulae, 1992

Table 2-1 presents a comparison of the parameters addressed by each of the design methods reviewed in this study.

Table 2-1 Summary of Design Parameters

Topic Addressed	Method						
	Rilem	DIN 4223	Danish DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
Bending	*	*		*			*
Shear (Unreinforced)	*	*	* (Walls)	*			
Shear (Reinforced)	*			*			
Axial Force	*		*	*		*	
Second Order Effects				*		*	
Anchorage	*	*		*			
Deflection Computations	*	*		*			
Deflection Limits	*	*		*			
Crack Formation	*		*	*			
Bearing Area	*		*		*		
Punching				*			
Torsion				*			

2.2 Rilem Recommended Practice

The Rilem Recommended Practice for AAC was put forth by the Rilem Technical Committee 78-MCA (Model Code for Autoclaved Aerated Concrete based on Rilem Test Methods) and 51-ALC (Test Methods for Autoclaved Lightweight Concrete). This model code, published in 1993, establishes a set of recommendations for the design of structures with AAC, and is based on relevant Rilem recommendations for the determination of material properties. Rilem presents methodologies for analysis of various failure modes. The failure modes included in this method are bending, shear, axial capacity, deflections, and anchorage. For each case, analytical expressions and design models are incorporated.

2.2.1 Design Philosophy

Rilem uses an ultimate strength design method, which is based on a semi-probabilistic approach that defines characteristic values for the loads while also applying safety factors.

2.2.2 Design Equations

2.2.2.1 Bending

For bending failure, four cases for analysis are presented. These situations correspond to increasing amounts of steel reinforcement and larger depths of the neutral axis. The details of the cross section are shown in Figure 2-1.

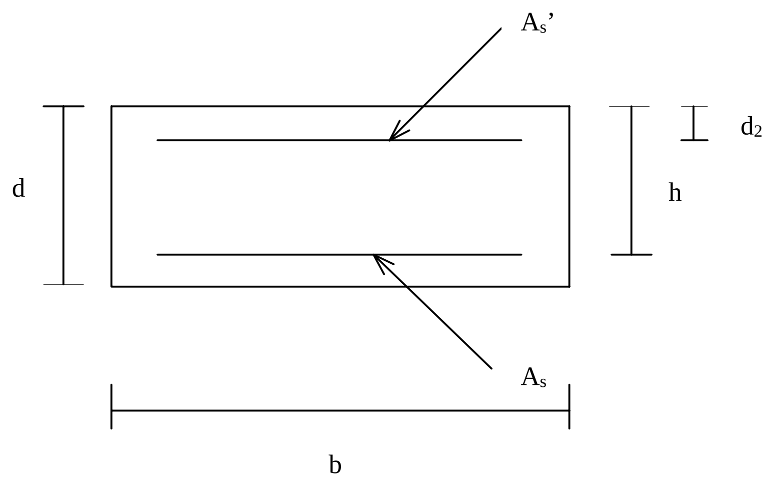

Figure 2-1 Nomenclature of Cross Section

where b is the width of the section, d is the depth of the section, h is the depth to the tensile reinforcement, d_2 is the depth of the cover for the compression steel, A_s is the area of longitudinal tensile reinforcement, and A_s' is the area of longitudinal compressive reinforcement.

Idealized bilinear stress-strain relations are used and are shown in Figure 2-2.

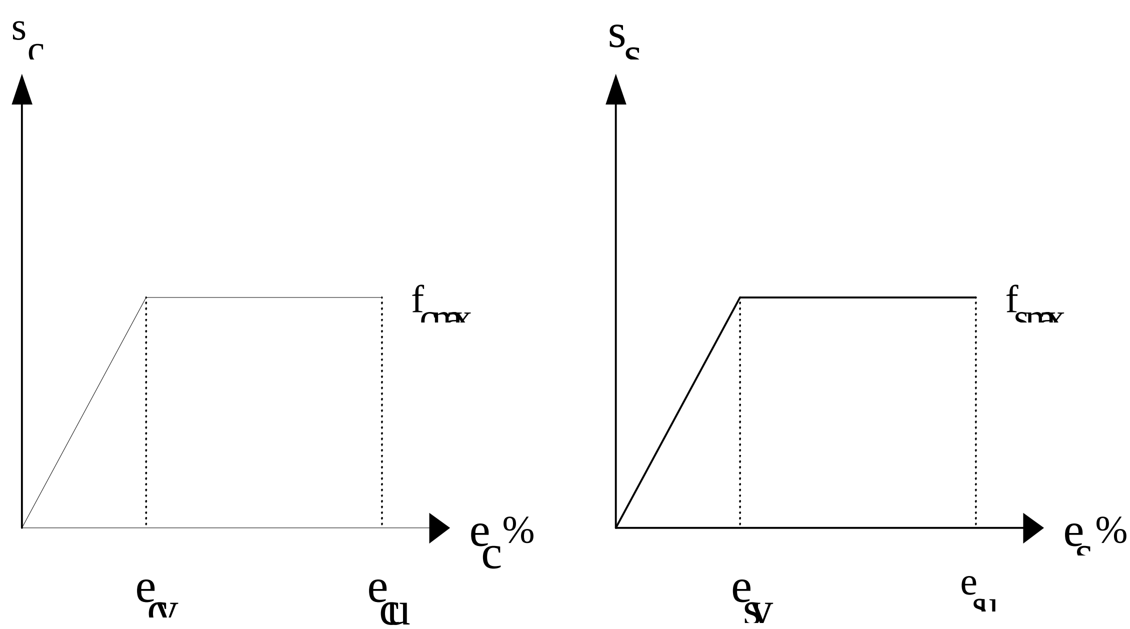

Figure 2-2 Compressive stress-strain relations for AAC and Steel

where σ_c is the stress of the concrete, σ_s is the stress of the steel reinforcement, ε_c is the strain if the concrete, ε_s is the strain of the steel reinforcement , ε_{cy} is the maximum elastic concrete compressive strain, ε_{su} is the ultimate steel tensile strain, and ε_{cu} is the ultimate concrete compressive strain.

The typical values for strain used in the bending failure calculations are as follows:

$\varepsilon_{cy} = 0.002$
$\varepsilon_{cu} = 0.003$
$\varepsilon_{su} = 0.005$

The Rilem code presents the equivalent to an ultimate stress approach, which is implemented throughout this report. The ultimate stress approach uses the following relationships with partial safety coefficients used as load factors in the calculation of the ultimate moment:

$$f_{c,\max} = f_{cd} = \frac{f_{ck}}{\gamma_c}$$

$$f_{s,\max} = f_{sd} = \frac{f_{sk}}{\gamma_s}$$

where $f_{c,max}$ is the maximum AAC compressive strength, f_{cd} is the actual design AAC compressive strength, f_{ck} is the characteristic AAC compressive strength, $f_{s,max}$ is the maximum steel tensile strength, f_{sd} is the actual design steel tensile strength, and f_{sk} is the characteristic steel tensile strength. The partial safety coefficients, γ_c and γ_s, are given as 1.4 for AAC in compression and 1.15 for steel.

Rilem also presents an approach using global safety coefficients for different types of failure. These global coefficients are used when the partial safety coefficients are equal to 1. Using the global coefficients for bending failure, the load bearing capacity, M_u, is found by the following relationships:

$$f_{c,\max} = f_{ck}$$

$$f_{s.\max} = f_{sk}$$

and

$$M_{per} = \frac{M_u}{\gamma_u}$$

where M_{per} is the permissible design moment and γ_u is the global safety coefficient for bending, taken as 1.8. This approach is not used for the design calculations in this report.

The four cases for analysis of bending failure are described herein.

<u>Case 1</u>:

In this case, a small amount of steel reinforcement is used, resulting in steel failure in tension. This failure may be represented by the following limit stresses:

$\sigma_c \leq f_{ck}$; $\sigma_s = f_{sk}$; $\sigma_s' = 0.75 f_{sk}$; $\varepsilon_c \leq \varepsilon_{cy}$; $\varepsilon_s = \varepsilon_{su} = 0.005$

where σ_s' is the maximum stress for steel in compression.

These stresses represent the situation of AAC in the linear range of the bilinear AAC stress-strain diagram and steel at the ultimate stress of the bilinear steel stress-strain diagram.

The stress-strain diagrams for Case 1 are shown below in Figure 2-3.

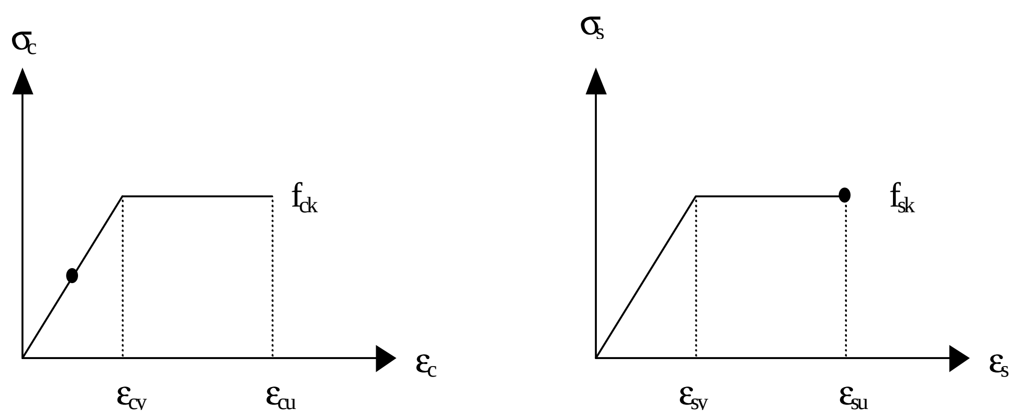

Figure 2-3 Stress-strain relation for AAC and steel for Case 1

Rilem presents the following equation to compute the position of the neutral axis:

$$s = -a_1 + \sqrt{\left(a_1^{\;2} + 2a_1\right)}$$

where $s = x_n / h$

$$a_1 = \left[(c - c')\cdot \varepsilon_{cy}\right] / \varepsilon_{su}$$

$$c = \left(A_s \cdot f_{sk}\right) / \left(bh \cdot f_{ck}\right)$$

$$c' = 0.75 \cdot c \cdot A_s' / A_s$$

where x_n is the depth of the neutral axis, h is the depth to the reinforcement, A_s is the area of longitudinal tensile reinforcement, b is the width of the section, and A_s' is the area of longitudinal compressive reinforcement.

Based on the above equations, the ultimate moment capacity may be computed as:

$$M_u = f_{ck} \cdot bh^2 \left[s^2 \cdot \frac{1 - s/3}{1 - s} \cdot \frac{\varepsilon_{su}}{2\varepsilon_{cy}} + c' \cdot \left(1 - \frac{d_2}{h} \right) \right]$$

This case is valid for the following conditions:

$$\varepsilon_c \leq \varepsilon_y$$

$$s \leq \frac{\varepsilon_{cy}}{\varepsilon_{cy} + \varepsilon_{su}} = \frac{0.002}{0.002 + 0.005} = 0.286$$

<u>Case 2:</u>

In this case, an intermediate amount of steel reinforcement is used, resulting in steel failure in tension with AAC falling into the nonlinear range. This failure may be represented by the following limit stresses:

$\sigma_c = f_{ck}$; $\sigma_s = f_{sk}$; $\sigma_s' = 0.75 f_{sk}$; $\varepsilon_{cy} \leq \varepsilon_c \leq \varepsilon_{cu}$; $\varepsilon_s = \varepsilon_{su} = 0.005$

The stress-strain diagrams for Case 2 are shown below in Figure 2-4.

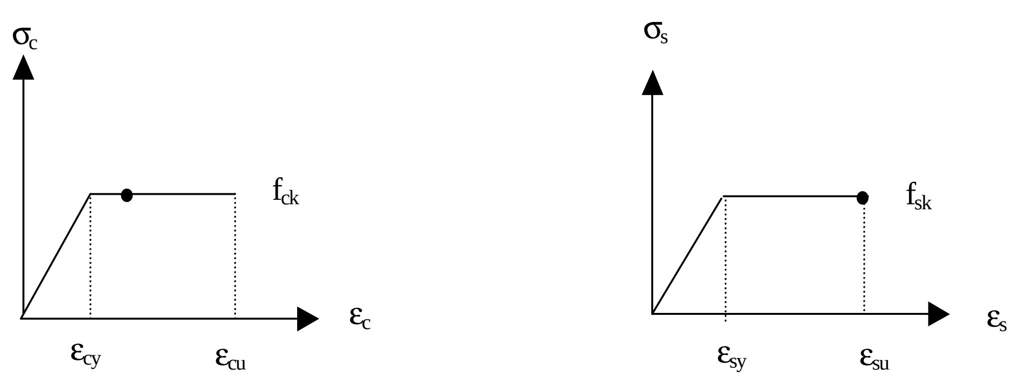

Figure 2-4 Stress-strain relation for AAC and steel for Case 2

Rilem presents the following equation to compute the position of the neutral axis:

$$s = \frac{k + c - c'}{1 + k}$$

where $\quad s = x_n / h$

$$k = \varepsilon_{cy} / 2\varepsilon_{su} \left(= 0..002 / 2 \cdot 0.005 = 0.2\right)$$

$$c = \left(A_s \cdot f_{sk}\right) / \left(bh \cdot f_{ck}\right)$$

$$c' = 0.75 \cdot c \cdot A_s' / A_s$$

Based on the above equations, the ultimate moment capacity may be computed as:

$$M_u = f_{ck} \cdot bh^2 \left[\alpha \cdot s \cdot (1 - \beta\, s) + c' \cdot \left(1 - \frac{d_2}{h}\right)\right]$$

where

$$\alpha = 1 - (1 - s)k / s \leq \alpha_{\max} = 0.667$$

$$\beta = \frac{2k(1 - s) \cdot \left[-1 + 2k(1 - s) / 3s\right] + s}{2s - 2k(1 - s)} \leq \beta_{\max} = 0.361$$

This case is valid for the following conditions:

$$0.286 \leq s \leq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon\varepsilon_{su}} = \frac{0.003}{0.003 + 0.005} = 0.375$$

<u>Case 3:</u>

In this case, a large amount of steel reinforcement is used, resulting in compressive failure of AAC. This failure may be represented by the following limit stresses:

$\sigma_c = f_{ck}$; $\sigma_s = f_{sk}$; $\sigma_s' = 0.75 f_{sk}$; $\varepsilon_c = \varepsilon_{cu} = 0.003$; $\varepsilon_{sy} \leq \varepsilon_s \leq \varepsilon_{su}$

The stress-strain diagrams for Case 3 are shown below in Figure 2-5.

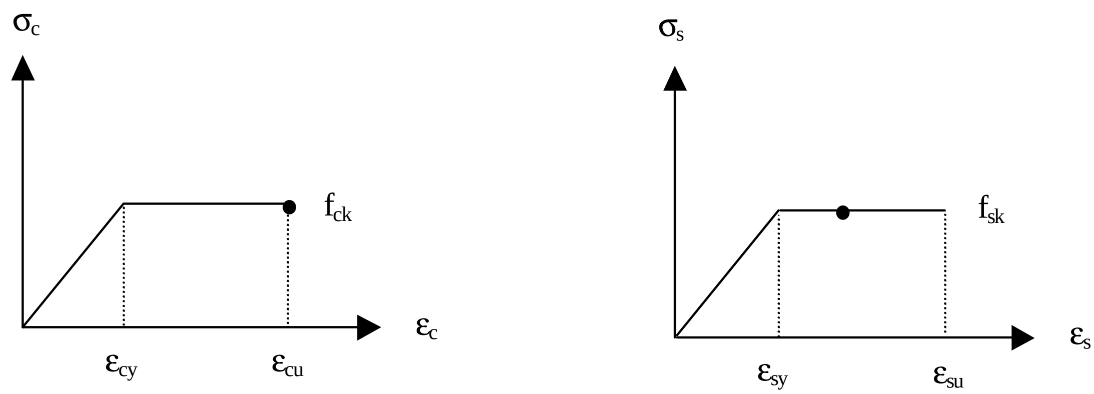

Figure 2-5 Stress-steel relation for AAC and steel for Case 3

Rilem presents the following equation to compute the position of the neutral axis:

$$s = \frac{c - c'}{a_{\max}}$$

$$\begin{aligned} where \quad & s = x_n / h \\ & c = (A_s \cdot f_{sk})/(bh \cdot f_{ck}) \\ & c' = 0.75 \cdot c \cdot A_s' / A_s \end{aligned}$$

where a_{max} is equal to 0.667 for $\varepsilon_{cy} = 0.002$ and $\varepsilon_{cu} = 0.003$.

Based on the above equations, the ultimate moment capacity may be computed as:

$$M_u = f_{ck} \cdot bh^2 [a_{\max} \cdot s \cdot (1 - \beta_{\max^s}) + c' \cdot (1 - \frac{d_2}{h})]$$

where β_{max} is equal to 0.361 for $\varepsilon_{cy} = 0.002$ and $\varepsilon_{cu} = 0.003$. This case is only valid for the following conditions:

$$0.375 \leq s \leq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} = 0.724$$

Case 4:

This case occurs when the AAC member is over reinforced. This case produces a sudden failure, therefore is not recommended by Rilem for practical design situations.

The following is a stepwise procedure for conducting a design example.

- Check Case 1 by determining $A_{s,max}$ using the following equation:

$$A_{s,\max} = \frac{1}{2} \times \frac{\varepsilon_{cy}}{\varepsilon_{cy} + \varepsilon_{su}} \times \frac{f_{ck}}{f_{sk}} \times bh + 0.75 A_s{}'$$

- If $A_s > A_{s,max}$; Try Case 2 (same procedure, etc.)

  If $A_s < A_{s,max}$; Determine the neutral axis by using the following equation:

$$s = -a_1 + \sqrt{\left(a_1{}^2 + 2a_1\right)}$$

where $s = x_n / h$

$$a_1 = \left[(c - c')\cdot \varepsilon_{cy}\right]/\varepsilon_{su}$$
$$c = (A_s \cdot f_{sk})/(bh \cdot f_{ck})$$
$$c' = 0.75 \cdot c \cdot A_s' / A_s$$

- Determine the bending capacity using the following equation with the value computed for s:

$$M_u = f_{ck} \cdot bh^2 \left[s^2 \cdot \frac{1 - s/3}{1 - s} \cdot \frac{\varepsilon_{su}}{2\varepsilon_{cy}} + c' \cdot \left(1 - \frac{d_2}{h}\right)\right]$$

2.2.2.2 Shear

Rilem presents two situations for the analysis of shear failure. These two cases involve considerations for members with and without shear reinforcement.

For members without shear reinforcement, the design depends on the concrete strength, the shear span ratio, and the amount of steel reinforcement.
The details of the cross section for shear strength analysis are shown below in Figure 2-6.

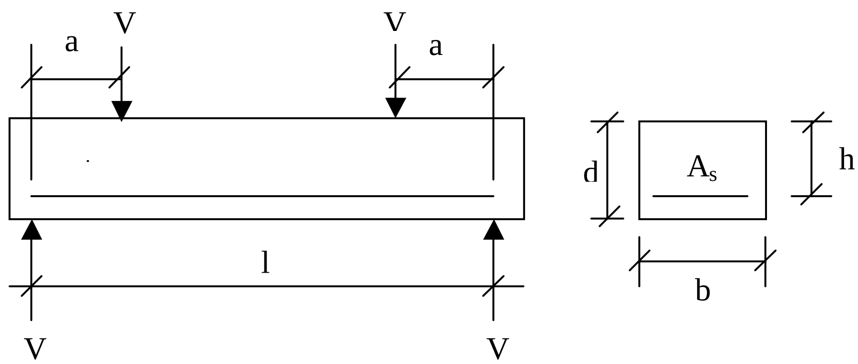

Figure 2-6 Details of Cross Section for Shear Strength Evaluation

where V is the shear force, l is the span of the slab, and a is the shear span.

The ultimate shear strength is calculated by the following equations:

$$\tau_u = 0.035f_k + 1.163\mu\ h/a - 0.053\ \text{MPa}$$

for $2.3 \le f_k \le 6.0$
$0.12 \le h/a \le 0.6,$
$0.12 \le \mu \le 0.8$

or: $\tau_u = 0.039f_k + 0.82\mu\ h/a - 0.075$ Mpa

if f_k, h/a, or μ fall outside the ranges presented above.

where τ_u is the nominal characteristic ultimate shear strength (MPa), f_k is the characteristic compressive strength of AAC (MPa), h and a are defined in Fig. 10, and μ is the ratio of tensile reinforcement ($\mu = 100A_s/b*h$).

For members with shear reinforcement the ultimate shear strength depends on the total percentage of shear reinforcement along with its yield strength. The following equation is used to calculate the ultimate shear strength for members with shear reinforcement:

$$\tau_u = 0.077f_k + 0.719\mu\ h/a + 0.001p_w f_{yw} - 0.142$$

and $p_w = p_{wi} + p_{wv};$
$p_{wi} = A_w\ ((\sin\theta + \cos\theta)\ 100\ /(b*s));$
$p_{wv} = A_w\ 100/(b*s)$

where p_w is the total ratio of shear reinforcement, p_{wi} is the ratio of inclined reinforcement, p_{wv} is the ratio of vertical reinforcement, A_w is the total area of shear reinforcement (mm^2), s is the spacing between shear reinforcement (mm), θ is angle of inclined shear reinforcement (degrees), and f_{yw} is the yield strength of shear reinforcement (MPa).

The shear span a, may be taken as l/4 for uniform loads. The shear capacity expression is limited to $b/h \leq 1.2$ and $s \leq h$.

2.2.2.3 Axial Force

In wall panels, the maximum capacity of an axially loaded unit can be found by testing. The whole unit can be tested and, in this case, the load bearing capacity is divided by a global coefficient, γ_u. Rilem presents an equation intended to limit tensile stresses on the wall by limiting maximum eccentricity. Rilem uses a uniaxial-bending interaction approach based on the Danish method, which is discussed later on in this report.

According to Rilem p. 64, an axially loaded column has to meet certain design criteria for calculation of the capacity. The criteria are as follows:

- The length between supports is considered the length of the wall.
- In analysis, the reduced effective length of the column due to added support can be taken into consideration.
- The design thickness of the column cannot be greater than the stated thickness, which is specified by the manufacturer or found by performance testing.
- The minimum width of single columns, with no reinforcement, cannot be less than the normal width used in testing.
- For wall units joined together, some units with smaller widths than usual can be used and no reduction of the maximum capacity is required.
- For panels that are axially loaded, the maximum capacity relies on the degree of the eccentricity. For calculations involving the maximum capacity, all moments and eccentricities should be considered.
- The reduced cross section should be taken into consideration in design, if there are holes and recesses in the unit.
- The average compressive strength of the AAC column at mid-height shall not surpass 20% of the characteristic compressive strength of the AAC.

The ultimate load bearing capacity may be calculated using the following equation:

$$R_d = \frac{l}{\gamma} \cdot R_t \cdot \frac{t_d - 2e_d}{t_d - 4e_t / 3} \cdot \frac{b_{eff}}{b_t} \cdot \frac{k_b}{k_{bt}}$$

Where R_d is the design load bearing capacity, γ_u is the safety coefficient, R_t is the characteristic load bearing capacity, t_d is the thickness of the cross section, e_d is the eccentricity due to the thickness, e_t is the declared maximum eccentricity, b_{eff} is the effective width of the cross section, b_t is the declared width, k_b (= 1/1(1+12 x 10^{-4} [l_d/(t_{bd} - 2e_d)^2])) is the slenderness factor due to the eccentricity e_d, k_{bt} (= 1/(1 + 12 x 10^{-4} [l_t/ (t_d - 4e_t/3)^2])), t_{bd} is the effective design thickness(= t_d for solid wall), l_d is the effective design length, and l_t is the stated unit length. The above equation for load bearing capacity is limited to the following:

$e_d < 2/3e_t$ and $0.85 < l_d < 1.15l_t$

2.2.3 Anchorage of Reinforcement

Anchorage of steel reinforcement in AAC according to Rilem is provided either by bond between the reinforcement and the concrete, or by crossbars, which are welded to the longitudinal bars and transfer the stresses by bearing. The bond and steel stress diagram is shown in Figure 2-7.

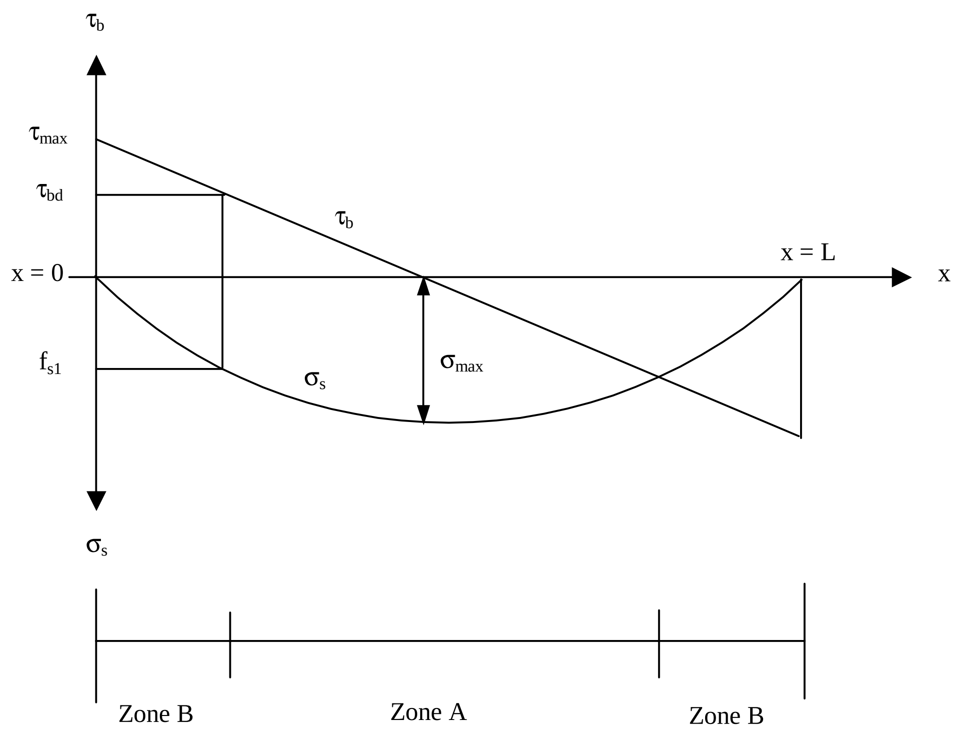

Figure 2-7 Bond and Steel Stress Diagram

where: $\tau_b \le \tau_{bd}$ = Zone A

$\tau_b \ge \tau_{bd}$ = Zone B

where τ_b is the bond stress from design loads, τ_{bd} is the design bond stress (= τ_k / γ_m), τ_k is the characteristic bond strength, and γ_m is a material safety coefficient for bond stress.

The characteristic bond stress for AAC is required by Rilem to be determined by a pullout test. Results are greatly influenced by the type of corrosive coating on the reinforcing and the class of the AAC.

In Zone B, where the design stresses in the steel exceed τ_{bd}, the steel pullout is resisted by crossbars, which are welded to the longitudinal reinforcement. These crossbars, which are placed perpendicular to the main longitudinal reinforcement, are located along the beam span using the following criteria:

Zone B:

- At least 50% of the required number of bars (n_a) (but not < 2) are to be placed at the supports along a length < 2 times the slab or beam thickness
- Distance between crossbars > 50 mm
- Distance between crossbars < slab depth.

Zone A:

- Distance between crossbars (not included in n_a calculation) < 1200mm.
- The number of crossbars for Zone B (n_a) in one reinforcing bar, calculated from the force F_a, is as follows:

$$n_a = \frac{F_a}{.28 f_k eks} \ge 2$$

$$n_a = \frac{f_{s1}}{f_d} \times 3$$

where:

k = 1 if $\,{}^{s}\!/\!{}_{d_a} \le 12$

k = 0.7 if $\,{}^{s}\!/\!{}_{d_a} = 20$

$$k = {}^{14d_a}\!/_{s} \quad \text{if} \quad {}^{s}\!/_{d_a} > 20$$

where e is the distance from the crossbar center line to the outside face of the concrete, s is the longitudinal reinforcement spacing, d_a is the cross bar diameter, f_k is the characteristic compressive strength of AAC, f_d is the steel design stress, and f_s is the stress in the steel at $\tau_b=\tau_{bd}$.

Anchorage calculations according to Rilem are based on an ultimate limit stress philosophy. The minimum load combinations recommended by Rilem for the analysis of anchorage are as follows:

Combination 1: 1.3 G + 1.5 Q
Combination 2: 0.9G + 1.4W
Combination 3: 1.2G + 1.2Q + 1.2W

Allowable material stresses are also factored. Bond stress used in conjunction with the above load factors is reduced by a factor of 2 in the final calculations to determine the number of anchorage bars.

The procedures presented in Rilem address uniformly distributed loads only. The effects of concentrated loads are not addressed.

2.2.4 Deflections Computations

Rilem provides a methodology to compute the deflections by considering the pre-cracking and post cracking state of the member. Rilem states that all deflection calculations are based on the relationship between the moment and curvature with the deflection, y, calculated as follows:

$$y = cl^2\, {}^{M}\!/_{EI}$$

where l is the span, c is a coefficient that varies due to loading and end conditions, and M, E, and I are the proper moment, modulus of elasticity, and moment of inertia. Rilem defines the general solution for deflections, which are based on permanent and short term loading. The relationship between permanent loads (q_p) and semi-permanent loads (q_t) is as follows:

$$m = {}^{q_p}\!/_{q_t}$$

Based on the above m factor (Rilem uses the variable c for the m factor), the effective modulus of elasticity and the effective tensile strength for AAC are defined as:

$$E_{eff} = E_c / (1 + c\phi)$$

$$f_{t(eff)} = f_{ct}(1 - c\phi)$$

where E_c is the short-term modulus of elasticity, f_{ct} is the tensile strength of AAC in bending, and ϕ is the creep factor of AAC recommended to be 1.0.

The bilinear diagram used to calculate deflections is shown in Figure 2-8.

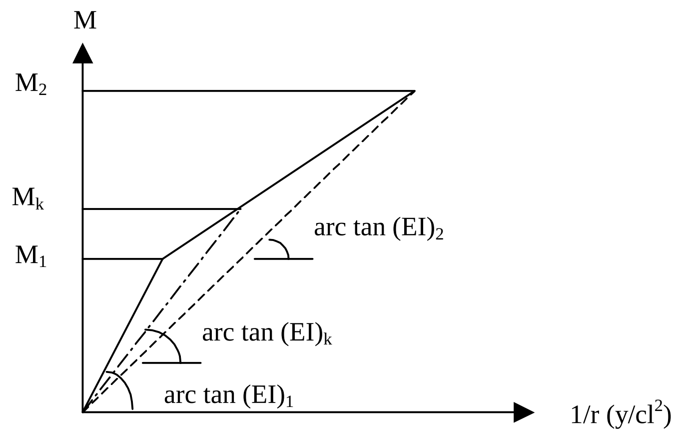

Figure 2-8 Bilinear Diagram for Estimation of Deflections

where M_1 is the cracking moment of the section where the tensile stress reaches $f_{t(eff)}$, $(EI)_1$ is the stiffness in the uncracked state using E_{eff}, M_2 is the ultimate bending moment of the section, $(EI)_2$ is the cracked section stiffness using E_{eff}, and M_k is the total design moment.

M_k sets the intersection, which defines the semi-cracked stiffness $(EI)_k$. Sag and deflections are calculated as follows using the above relationships:

$$Sag = y_k - y_s - y_0$$
$$\text{Active Deflection} = y_k - y_i$$

where y_k is the deflection from the total load using $(EI)_k$, y_s is the instantaneous deflection resulting from the short-term load using a stiffness for the section defined by:

$$(EI)_0 \frac{(EI)_k}{(EI)_1}$$

where $(EI)_0$ is the uncracked section stiffness based on the short term modulus of elasticity of AAC as defined by Rilem, y_0 is equal to the camber of the section, and y_i is the instantaneous deflection under loads occurring before the attachment of adjourning members, assuming an uncracked state and the short term modulus of elasticity.

2.2.5 Deflection Limits

Rilem approaches deflections as a serviceability limited state condition. Rilem controls deflections and cracking under service conditions and working service loads by limiting deformations, which affect the use or appearance of the structure, and crack formation, which affects the appearance and durability of the structure.

For roof and floor slabs the final sag, which is defined as long-term deflection v, is limited to:

$v < L/250 < 30$ mm

Short-term deflection, v_0, due to full-superimposed loading is limited to:

$v_0 < L/400 < 20$ mm.

where L is the span of the member.

For roof and floor slabs supporting or attached to non-structural elements, which can be damaged by deflection, the active deflection, v_a, is limited to:

$v_a < L/450$

where v_a is defined as the part of the total deflection, which occurs after non-structural elements are attached.

2.2.6 Crack Formation

Rilem recommendations for controlling and limiting section cracking are intended to control the corrosion of the reinforcement and are for aesthetical concerns. Rilem specifies that at least one of the following criteria be satisfied:

- Satisfactory performance, which limits the maximum crack width to 0.2 mm at total load.
- $d_b < .2(100\mu + 8)$;

where d_b is the diameter of the longitudinal reinforcement, and μ is the percentage of reinforcement.

- 1.5 d $\frac{\sigma_s}{E_s}$ < 0.35

  where d is the depth of the slab, σ_s is the tensile steel stress (Mpa), and E_s is the modulus of elasticity of steel (Mpa).
- Testing results that show corrosion resistance is sufficient at stresses in excess of working/service load levels.

2.2.7 Bearing

Rilem presents an equation to compute the minimum area of support to prevent excessive bearing stresses for vertical actions. This equation is based on the Danish method, which will be discussed later in this report. The wall strength should not be assumed to be greater than the strength at the edge of the unit. The edge strength, $f_{c,edge}$, is equal to 0.7 f_{ck}. The equation for design bearing capacity is as follows:

$$R_{bd} = (f_{c,edge} \times b_p \times a_p \times k_1 / \gamma_u)$$

where R_{bd} is the design bearing capacity at the supports, b_p is the dimension of the support parallel to the plane of the unit, a_p is the dimension of the support perpendicular to the plane of the unit, k_1 is a coefficient (= $0.2 + 0.6 \sqrt{(A_2/A_1)} < 5$), A_1 is the effective area of bearing, A_2 is the centrally loaded area which occurs with equal distribution of the vertical action, and γ_u is the global safety coefficient.

2.2.8 Summary

The Rilem recommended practice is intended for the design of reinforced AAC panels in bending, compression, and shear, and includes provisions for the evaluation of deflections and also recommendations for serviceability conditions such as, deflection limits and cracking limitations. All the information is presented in SI units. The Rilem recommended practice should be used within an ultimate state design approach, although, as an alternative, it includes the appropriate load and material factors if the design is to be conducted using global safety coefficients. Rilem addresses bending by defining four different failure modes that determine the proper equations to be used for evaluation of the capacity of the panel in bending. The modes addressed may be stated as follows:

- Case 1: Tensile failure of reinforcement with AAC in the elastic range
- Case 2: Tensile failure of reinforcement with AAC in the inelastic range
- Case 3: Compressive failure of AAC with reinforcement in the inelastic range
- Case 4: Compressive failure of AAC with reinforcement in the linear range.

Based on the applicable failure case, Rilem provides different design equations for the evaluation of the bending capacity of the AAC panel. Those design equations are based on equilibrium of forces acting on the cross section considering a triangular or trapezoidal stress distribution on the compression side of the AAC when subjected to the bending stresses. The equations also include considerations for the longitudinal steel in tension.

Rilem recommended practice includes design equations for the evaluation of the shear capacity of members with or without shear reinforcement. Rilem also contains design equations for the evaluation of the load bearing capacity of axially loaded members. The equation presented allows determination of the load bearing capacity of members subjected to axial and bending stresses simultaneously, however, in the determination of the load bearing capacity, the longitudinal reinforcement of the panel is not considered. Also, the load bearing capacity equations do not incorporate considerations for second order effects.

Rilem recommended practice presents equations for the evaluation of the anchorage capacity of reinforcement. The equations presented allow limited consideration of bonding between AAC and reinforcement. However, Rilem recommends that in critical locations, bonding effects should be ignored in such a way that the anchorage capacity is computed based on the anchorage provided by welded crossbars bearing on AAC only.

Rilem also presents design equations to check bearing stresses at supports of floor panels or where point loads could induce bearing failure.

Rilem provides serviceability criteria based on deflection limits and cracking limits. For the computation of deflections, the use of the bilinear diagram, which allows an estimation of deflections of the reinforced panel before and after cracking is recommended.

Rilem does not provide any particular provision regarding the use of fly ash based AAC for reinforced panels.

2.3 DIN 4223 Guidelines for Reinforced AAC Roof and Ceiling Slabs

The German standards addressing AAC are contained in DIN 4223 “Reinforced Roof and Ceiling Panels from steam-cured aerated concrete and foamed concrete”, which was originally published in 1958. This standard presents provisions for the calculation, manufacture, use, and testing of AAC for floor and roof panels. The DIN standard contains methodologies for analysis of bending, shear, deflections, anchorage, and crack formation.

2.3.1 Design Philosophy

DIN 4223 uses an allowable stress design method including reduction factors. The factors increase the safety against failure and are based on the static and live loads for the maximum bending moment multiplied by 1.75.

2.3.2 Design Equations

2.3.2.1 Bending

DIN 4223 presents bending equations based on elastic theory, and assumes a parabolic stress distribution for compressive stresses in AAC. Using an allowable stress method, the ultimate bearing capacity is found by multiplying a safety factor of 1.75 times the largest bending moment, M_{g+p}, calculated from dead and live loads.

DIN standards are based on the assumption that plane sections remain plane, with the elongation of the steel, ε_e, taken as 0.002, and the maximum compressive strain of concrete, $\varepsilon_{b,max}$, also taken as 0.002.

The stress in the reinforced AAC is represented by a parabolic distribution. The stress and strain distributions are shown in Figure 2-9.

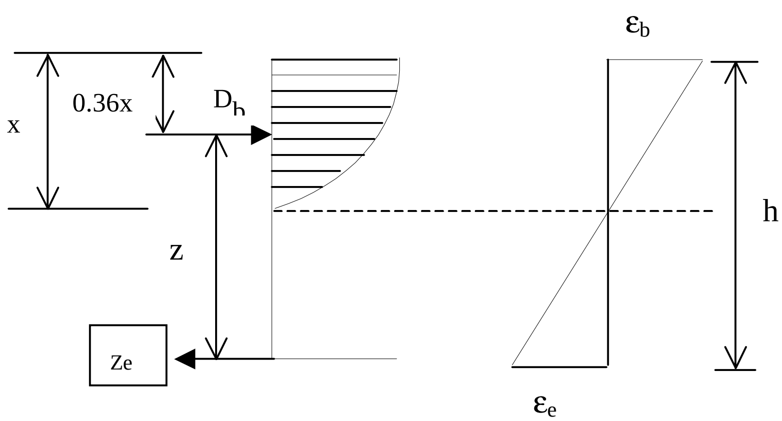

Figure 2-9 Stress-Strain Distribution for AAC in Bending

In the stress distribution diagram, the variable z represents the lever arm of the internal forces. The location of the centroid is a distance 0.36x from the pressure edge, so $z = h - 0.36x$, where h is the height of the slab in cm and x is the distance of the neutral axis from the pressure edge in cm. Also, Z_e represents the tensile force of the steel and ε_e represents the maximum strain of the steel.

The compressive force, D_b, which can be absorbed by a rectangular bending compression zone, is represented by the following equation:

$$D_b = 0.60 \times b \times x \times \frac{2}{3} \times W \times \frac{\varepsilon_b}{\varepsilon_{b,\max}}$$

where b is width of the compression zone in cm, W is the compressive strength of concrete which is obtained from Table 1, Column 3 in DIN 4223, and ε_b is the strain of the concrete at the pressure edge.

DIN 4223 focuses on procedures for design of slabs, rather than analysis. Therefore, based on the above conditions, DIN 4223 presents coefficients for dimensioning a slab with a given value for M_{g+p} in kg-m. These coefficients, k_h and k_z, are functions of the concrete compression. The coefficients take into account a global safety factor of 2.625 for steel and concrete. This factor includes a safety factor of 1.75 for steel and 50% higher in the concrete.

The height, h, of the slab and the lever arm, z, are found using the following equations:

$$h = k_h \sqrt{\frac{M_{g+p}}{b}}$$

$$z = k_z \times h$$

DIN 4223 also provides an equation for determination of the steel reinforcement:

$$F_e = \frac{M_{g+p}}{z \times \sigma_{ezul}}$$

where F_e is the required cross-sectional area of the slab and σ_{ezul} is the permissible steel stress.

2.3.2.2 Shear

DIN 4223 provides a check for shear stresses in members without shear reinforcement. The equation for determining the shear stress, τ_o, is:

$$\tau_0 = \frac{Q}{b \times z}$$

where Q is the highest transverse force under continuous and live loads in kg, b is the slab width in cm, and z is the lever arm of the internal forces in cm.

DIN 4223 limits shear stresses to according to the quality grade based on the compressive strength of AAC. The limits may be stated as follows:

For grade GSB 35: 0.8 kg/cm^2
For grade GSB 50: 1.2 kg/cm^2

2.3.2.3 Axial Force

DIN 4223 does not provide information on axially loaded AAC panels.

2.3.3 Anchorage of Reinforcement

For anchorage of the steel reinforcement, DIN 4223 provides an equation to compute the number of transverse bars as follows:

$$n = \frac{Z^2}{2500 d_1 \times W}$$

where Z is the tensile force of the steel reinforcement, d_1 is the diameter of the cross bars in cm, and W is the compressive strength of concrete in kg/cm^2.

The force, Z/n, which is transferred by the welding point, may not be larger than 1/3 of the force P, which is as follows:

Reinforcing steel I: $P = 0.50 * F_{el} * \sigma_s$

Reinforcing steel IV: $P = 0.35 * F_{el} * \sigma_s$

where reinforcing steel I or IV are to be used as reinforcement according to DIN 1045, F_{el} is the diameter of the longitudinal rods, and σ_s is the minimum tensile yield point.

2.3.4 Deflection Computations

DIN 4223 presents equations for computing deflections based on elastic analysis. These equations are as follows:

For strength class GSB 35: $l \leq 160 \times d \times \sqrt[3]{\frac{c}{q}}$

For strength class GSB 50: $l \leq 175 \times d \times \sqrt[3]{\frac{c}{q}}$

where l is the span in cm, d is the panel thickness, c is a coefficient based on the percentage of reinforcement ,μ_d, which is given in Table 3 in DIN 4223 and panel thickness, and q is the full calculated live load in kg/m^2.

2.3.5 Deflection Limits

DIN 4223 states that deflections should be limited to span/300 for members in bending using the maximum bending moment from the static and live loads.

2.3.6 Crack Formation

DIN 4223 provides no information on crack formation.

2.3.7 Summary

DIN 4423 presents design provisions for reinforced AAC panels used as slabs. The information is presented in MKS units, and the design equations are intended for use with an allowable stress design approach. DIN 4223 provides equations for the evaluation of the bending and shear capacity of reinforced AAC panels, as well as the anchorage strength of longitudinal reinforcement. DIN 4223 also provides equations to compute deflections and provides serviceability checks based on deflection limits. The bending capacity of the reinforced panel is evaluated by equilibrium of internal forces acting on the cross section in bending. In computing the internal forces, a parabolic distribution of stresses in the compression side is assumed, and the contribution of the reinforcement in compression is ignored.

DIN 4223 presents an equation for the evaluation of the shear strength of panels only in bending without shear reinforcement. Reinforced bending members with shear reinforcement are not directly addressed in DIN 4223. The anchorage capacity of the longitudinal reinforcement is evaluated by considering only the effect of transverse bars welded to the longitudinal bars. The contribution of bonding to the anchorage capacity is neglected.

DIN 4223 provides an indirect method for the computation of deflections. The indirect method is intended to determine the appropriate dimensions of the panel in such a way that when fully loaded, the deflections do not exceed the deflection limits specified by DIN 4223. The indirect method is based on elastic theory and therefore, does not address the computation of deflections after cracking.

DIN 4223 also presents design equations to check bearing stresses at supports or where point loads could induce bearing failure.

DIN 4223 does not provide any particular provision regarding the use of fly ash based AAC for reinforced panels.

2.4 Danish Standard DS 420 Code of Practice for the Structural Use of Lightweight Concrete

Danish Standard DS 420 covers the use in buildings of reinforced, industrially produced members, deck and roof elements, and laterally loaded horizontally mounted walls. DS 420 presents methodologies for analysis of bending, shear, axial capacity, crack formation, and supports.

2.4.1 Design Philosophy

DS 420 uses an ultimate strength design approach that includes material and load factors to establish the ultimate limit states.

2.4.2 Design Equations

2.4.2.1 Bending

DS 420 relies on the characteristic strengths given in the manufacturer's specifications for bending and therefore does not provide design equations for the evaluation of bending capacity of AAC members.

2.4.2.2 Shear

For the evaluation of shear capacity, DS 420 provides equations that are applicable only to axially loaded members with in-plane lateral loading (shear walls). The equations are as follows:

$$V_d = \mu N_d + cA_c$$

for: $0.02 < \sigma_d/f_{cg} < 0.3$

and $$V_d = (\mu 0.02 f_{cg} + c) \times A_c \times \frac{\sigma_d}{0.02 f_{cg}}$$

for: $\sigma_d < 0.02\ f_{cg}$

where σ_d is the design normal stress of the concrete, f_{cg} is the declared compressive strength of the concrete, V_d is the design load-carrying capacity of the joint in shear, μ is the design coefficient of friction , c is the design cohesion, and A_c is the compressed area. The values for μ and c are given in Table V 6.2.7 in DS 420.

2.4.2.3 Axial Force

DS 420 provides equations for the calculation of the load-carrying capacity of columns due to axial forces. It also states that when determining the load-carrying capacity, the column effect and any eccentricities should be taken into account.

The design load-carrying capacity, R_{sd}, is calculated as follows:

$$R_{sd} = k_s \times \frac{f_{cg}}{\gamma_m} \times A_c$$

where γ_m is a partial safety coefficient and k_s a column factor and is written as follows:

$$k_s = \frac{1}{1 + \frac{f_{cg}}{E_{ok}\pi^2} \times \left(\frac{l_s}{i_c}\right)^2}$$

which, unless otherwise noted, may be written as:

$$k_s = \frac{1}{1 + 12 \times 10^{-4} \times \left(\frac{l_s}{t_s - 2e_t}\right)^2}$$

where E_{ok} is the characteristic initial modulus of elasticity, l_s is the free length of the column, i_c is the radius of gyration for the compressive zone of the cross-section, t_s is the design thickness of the column, and e_t is the resulting eccentricity perpendicular to the wall.

If the following conditions are satisfied:

$e_t < 2/3\ e_g$ and $0.85\ l_{sg} < l_s < 1.15\ l_{sg}$

the design load carrying capacity, R_d, can be written as follows:

$$R_d = \frac{1}{\gamma_m} \times R_g \times \frac{t - 2e_t}{t - \frac{4}{3}e_g} \times \frac{b_e}{b_g} \times \frac{k_s}{k_{sg}}$$

where e_g is the declared eccentricity at the top of the column, l_{sg} is the declared column length, R_g is the declared element strength, b_g is the declared element width, and k_s and k_{sg} are as follows:

$$k_s = \frac{1}{1 + 12 \times 10^{-4} \times \left(\frac{l_s}{t_s - 2e_t} \right)^2}$$

$$k_{sg} = \frac{1}{1 + 12 \times 10^{-4} \left(\frac{l_{sg}}{t - \frac{4}{3}e_g} \right)^2}$$

These equations are identical to those provided by Rilem for axially loaded units.

2.4.3 Anchorage of Reinforcement

DS 420 does not contain information on anchorage of steel reinforcement in AAC.

2.4.4 Deflection Computations

DS 420 does not contain information on deflection computations.

2.4.5 Deflection Limits

DS 420 does not contain information on deflection limits.

2.4.6 Crack Formation

DS 420 limits the crack width in beam elements to 0.2 mm for the serviceability limit state.

2.4.7 Bearing

DS 420 states that for vertical actions the wall strength should not be considered greater than the compressive strength of the concrete at the wall's edge. The equation presented to account for bearing stresses is as follows:

$$R_d = \frac{1}{\gamma_m} \times (f_{c,edge}) \times b \times a \times k_1$$

where

$$k_1 = 0.2 + 0.6\sqrt{\frac{A_2}{A_1}}$$

where $f_{c,edge}$ is the compressive strength of the concrete at the wall's edge, a is the dimension of the support perpendicular to the plane of the wall, A_1 is the effective bearing area, and A_2 is the centrally loaded area.

2.4.8 Summary

The Danish Standard DS 420 presents provisions for the structural design of AAC walls. The provisions are presented in mandatory language and the information is presented in SI units. The design equations presented are intended for use within an ultimate state design approach. DS 420 relies on manufacturer's specifications for characteristics strengths of AAC and does not address bending analysis. DS 420 addresses AAC panels in shear in a limited manner by specifying that the loads applied should generate shears that do not exceed the design capacity of the member. For members under axial load, DS 420 presents equations for the evaluation of the load bearing capacity of the member without consideration for the longitudinal reinforcement or second order effects. These equations are identical to those presented in Rilem Recommended Practice. Also, DS 420 presents equations for the evaluation of shear capacity of walls under combined axial and lateral loads acting on the plane of the wall. These design equations ignore the effect of shear reinforcement in the wall. DS 420 also presents design equations to check bearing stresses at supports or where point loads could induce bearing failure. These design equations are the same as those presented by Rilem.

DS 420 does not directly address serviceability, except by recommending that the crack width be limited to 0.2 mm for beam elements under service loads.

DS 420 does not provide any particular provision regarding the use of fly ash based AAC for reinforced panels.

2.5 Draft European Standard prEN12602, "Prefabricated Reinforced Components of Autoclaved Aerated Concrete"

European Standard PrEN12602 is a draft of a technical standard published in 1996 intended to address the design of prefabricated reinforced autoclaved aerated concrete. The European Committee for Standardization - technical committee CEN/TC 177, developed this draft standard and it is currently under review by this committee for adoption as a technical standard.

PrEN12602 covers the following topics related to AAC:

- Properties and Requirements
- Guidance for the design of AAC components
- Basis for AAC design
- Design by calculations
- Design by testing

For the purpose of this study, the focus of the review of PrEN12602 is on the chapters with provisions for the design of AAC components, basis for AAC design, and design by calculations.

PrEN12602 presents a discussion about constituent materials of AAC, general properties of AAC, such as dry density, compressive strength, tensile strength, stress-strain diagrams, modulus of elasticity, Poisson's ratio, coefficient of thermal expansion, shrinkage, creep, thermal conductivity, bond to reinforcement, and thermal prestress. PrEN12602 includes dimensions and tolerances of AAC members, as well as minimum thickness and maximum slenderness requirements for reinforced AAC components.

The minimum thickness required for AAC components according to this standard is 30 mm, and the maximum slenderness components are shown in Table 2-2, where L is the free span between the edges of the supports and h is the depth of the cross-section.

Table 2-2 Maximum Slenderness for Reinforced AAC Members

Member	L/h
Roof Components	40
Floor Components	30
Wall Components (subjected to external axial load)	35
Other Wall Components	45
Beams	20

PrEN12602 presents minimum requirements for reinforcement. These requirements include maximum and minimum spacing for longitudinal and transverse bars, bend diameters, and support length for anchorage purposes.

2.5.1 Design Philosophy

PrEN12602 addresses the design of AAC components such as roof and floor panels, beams, non-load bearing walls, and bearing walls. For roof and floor panels, the European standard recommends solid, simply supported components loaded in one-way only and with single or multi-layer reinforcement. For beams, PrEN12602 recommends simply supported components with longitudinal and vertical, or diagonal shear reinforcement. For non-load bearing walls, PrEN12602 recommends considerations of dead loads, and in the case of external walls, it recommends wind loads for the design of this type for structural members. For load bearing walls, PrEN12602 recommends considering the overall stability of the member, and axial, bending, and shear actions including buckling effects and accidental eccentricities.

PrEN12602 accepts two different design methodologies, which are design by calculation and design by functional testing of structural components. The design is based on checks for the ultimate and serviceability limit state.

The design by calculation requires considerations for the ultimate and serviceability limit state. For the ultimate limit state, PrEN12602 presents provisions for bending, axial compression, shear, buckling, punching, and torsion combined with shear. For the serviceability limit state, PrEN12602 presents provisions for stress limitations, crack control, and deflections control.

PrEN12602 presents a serviceability limit state of stress limitations in order to prevent the formation of longitudinal cracks or micro-cracks on AAC under service load conditions, and in order to control higher than predicted levels of creep. In order to prevent longitudinal cracks, PrEN12602 recommends limiting compressive stresses under service loads to a maximum of $0.6f_{ck}$. In structures where creep may affect the functionality of the structure, such as cantilevered members, it is recommended that compressive stresses on AAC do not exceed $0.45f_{ck}$. To prevent excessive deformation of reinforcing steel, it is recommended that tensile stresses in reinforcement do not exceed $0.8f_{syk}$.

2.5.2 Design Equations

2.5.2.1 Bending

For the evaluation of the bending capacity of AAC members, PrEN12602 uses the following bilinear stress-strain diagrams for AAC and steel respectively:

For AAC:

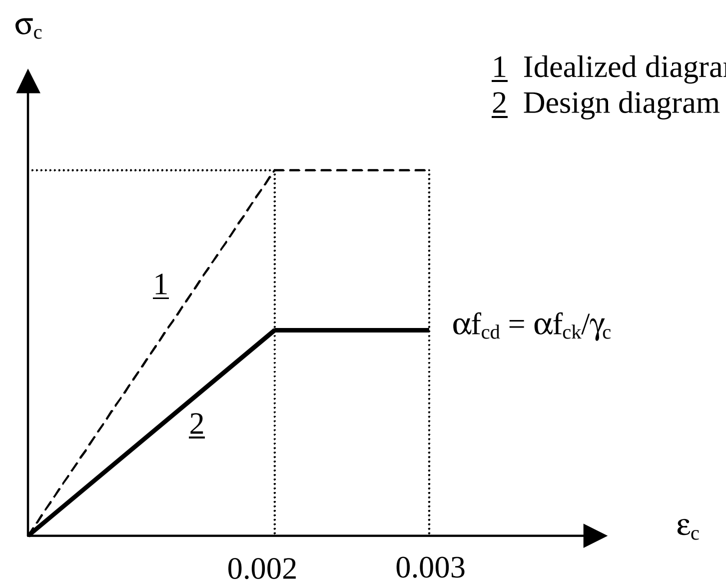

Figure 2-10 Bilinear Stress-Strain Diagram for AAC

where the design value of the compressive strength of AAC is defined by:

$$f_{cd} = f_{ck} / \gamma_c$$

where f_{cd} is the design value of the compressive strength of the AAC, f_{ck} is the characteristic strength of the AAC, γ_c is the partial safety factor for AAC, and α is a coefficient which takes into account the long term effects of the compressive strength and the unfavorable effects resulting from the way the load is applied.

For steel:

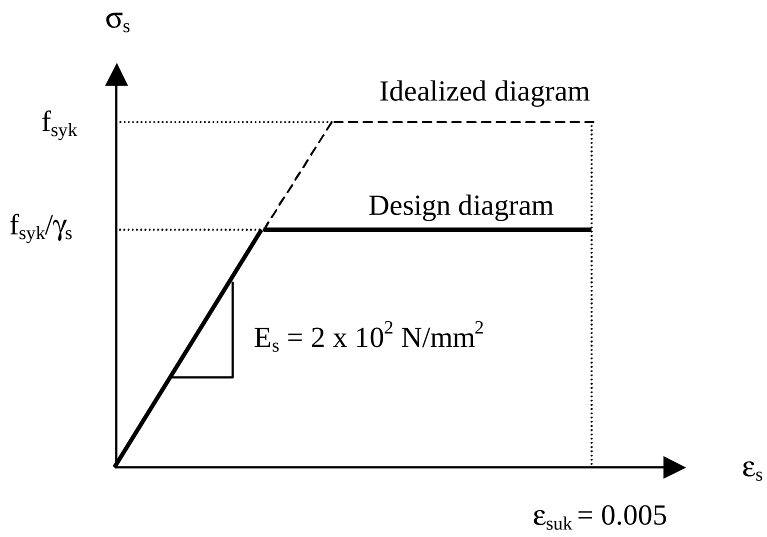

Figure 2-11 Stress-Strain Diagram for Steel

where f_{yk} is the characteristic yield stress of reinforcing steel, γ_s is the partial safety factor for reinforcing steel, and E_s is the modulus of elasticity of reinforcing steel. The bending capacity of the member is found by equilibrium of internal forces in the cross section assuming the following strain distributions:

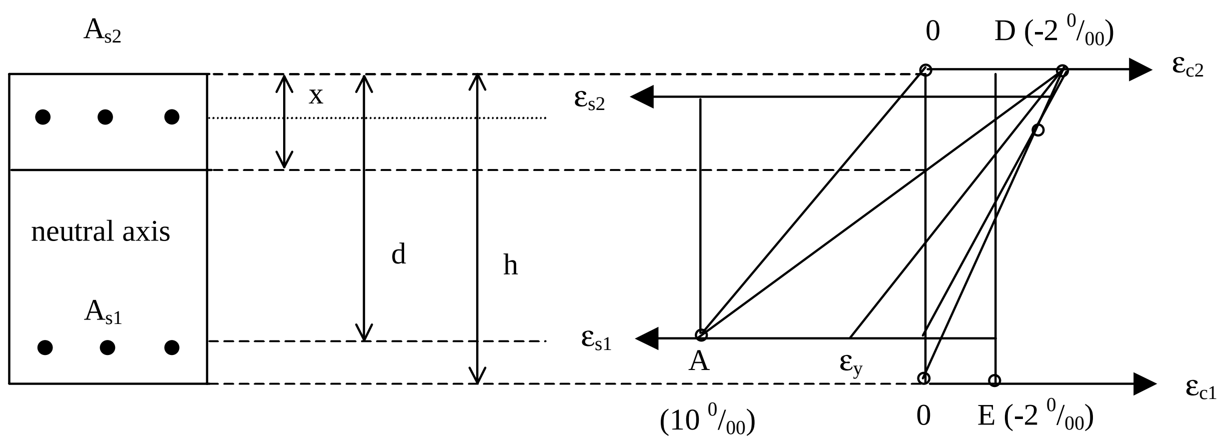

Figure 2-12 Accepted Strain Distributions for Reinforced AAC

In the evaluation of the bending capacity of the AAC member, the tensile strength of AAC is neglected, and it is assumed that compatibility of strains exists between the reinforcement and the surrounding AAC material. In order to prevent brittle failure, PrEN12602 presents a provision for a minimum amount of reinforcement in such a way, that the bending capacity of the AAC member is always greater than the cracking moment of the cross section.

2.5.2.2 Shear

PrEN12602 includes a detailed methodology for the evaluation of the shear capacity of AAC members. This methodology considers the cases of members under transverse loads with or without shear reinforcement, and members under centric or eccentric axial loads.

PrEN12602 defines three different parameters to evaluate the shear strength of reinforced AAC members. V_{Rd1} is the design shear strength for members without shear reinforcement, V_{Rd2} is the maximum design shear force that can be applied on the member without crushing of the notional AAC compressive struts, and V_{Rd3} is the design shear force that can be carried by a member with shear reinforcement. The criterion for shear design is as follows. If the design shear force is less than V_{Rd1}, no shear reinforcement is required. If the design shear force exceeds V_{Rd1}, shear reinforcement should be provided in such a way that the design shear force does not exceed V_{Rd3}. The design shear force should not exceed V_{Rd2} under any circumstances.
The expressions for V_{Rd1}, V_{Rd2}, and V_{Rd3} are:

$$V_{Rd1} = \frac{T_{Rd}}{\gamma_n}(1-0.83d)(1+240\rho_1)b_w d$$

$V_{Rd2} = 0.5 v f_{cd} b_w 0.9d$ for members without shear reinforcement, and

$V_{Rd2} = 0.17 a f_{cd} b_w z$ for members with vertical shear reinforcement.

$V_{Rd3} = V_{Rd1} + V_{wd}$ with

$$V_{wd} = \frac{A_{sw} z \sigma_{swd}}{s}$$

where

$$T_{Rd} = 0.063\frac{f_{ck}^{0.5}}{\gamma_C}$$

and

f_{ck} = Characteristic compressive strength of AAC
γ_C = partial safety factor for AAC
γ_n = multiplication factor for AAC, taken as 1.2
d = effective depth of the section in meters
ρ_1 = reinforcement ratio
b_w = minimum width of the section over the effective depth

$$v = \frac{0.7 f_{ck}}{30} \geq 0.5$$

A_{sw} = cross-sectional area of vertical shear reinforcement
S = spacing of shear reinforcement

$$\sigma_{swd} = \frac{30}{\gamma_c} f_{ck} \frac{\phi_{si}^2 + \phi_{sw}^2}{\phi_{sw}^2}$$

ϕ_{si} = diameter of longitudinal bars
ϕ_{sw} = diameter of shear reinforcement
z = 0.85d

PrEN12602 considers those portions of the cross sectional area that are not subjected to tensile stresses only effective for shear. The shear capacity of the member is evaluated as follows:

$$V_{Rd} = T_{Rd} b_w \frac{x}{\gamma_n}$$

where

$$T_{Rd} = 0.06 \frac{f_{ck}^{0.5}}{\gamma_c}$$

and x is the depth of the neutral axis.

PrEN12602 includes provisions to evaluate the interaction of torsion and shear in members with and without torsion reinforcement. For the case of members requiring torsion reinforcement, PrEN12602 presents equations to compute the required amount of steel to support shear stresses induced by torsion.

PrEN12602 includes provisions for the evaluation of the punching strength of AAC members subjected to concentrated loads. Those provisions are based on the computation of the critical area subjected to punching stresses and the shear strength of the AAC member.

2.5.2.3 Axial Force

For the evaluation of the axial capacity of reinforced AAC members, PrEN12602 uses the equilibrium of internal forces, considering the strain-stress diagrams shown in Figure 2-10 and Figure 2-11, and the strain distributions shown in Figure 2-12. PrEN12602 recommends ignoring the compression steel for the evaluation of the axial capacity of reinforced AAC members unless the reinforcement is sufficiently restrained against buckling by some mechanical means, such as stirrups.

PrEN12602 includes two methods for the consideration of slenderness effects on vertical load bearing AAC components not exceeding the slender limits presented in Table 2-2. The first method is based on the Euler formula, and it is intended for AAC components loaded only by centric or eccentric forces, neglecting the reinforcement.

In this case, the design axial load capacity may be determined as follows:

$$N_{Rd} = k_s a f_{cd} A_c$$

with

$$k_s = \frac{1}{\left[1 + \left[\frac{f_{cd}}{E_{cm}\pi^2}\right] x \left[\frac{L_o}{i_c}\right]^2\right]}$$

where

E_{cm} = modulus of elasticity of AAC
L_o = effective length of the member
i_c= radius of gyration of the compression zone
F_{cd}= design compressive strength of AAC
a = coefficient for long term effects, taking as 0.85

The second method is intended for load bearing AAC members classified as slender, isolated columns subjected to axial and lateral loads. This method requires iteration to consider second order effects, or the use of approximate formulas for the evaluation of second order effects.

2.5.3 Anchorage of Reinforcement

Anchorage of steel reinforcement is required in such parts of the structural member where the bond stress under the design load exceeds the design bond strength. The number and distribution of transverse bars is determined as follows:

$$F_{RA} \geq F_{lD}$$

where

F_{RA} = anchorage capacity of transverse anchorage bars
F_{lD} = tensile force in longitudinal bars

F_{RA} may be computed as:

$$F_{RA} = 0.83 n_t \phi_t t_t f_{ld} \leq 0.8 n_t \frac{F_{wg}}{\gamma_C}$$

where

F_{wg} = declared shear strength of welded joint
n_t = number of transverse bars between the critical section and the end of the member
ϕ_t = diameter of transverse anchorage bars
t_t = effective length of transverse bars, equal to distance between longitudinal bars, but not greater than $14\phi_t$.
f_{ld} = design bearing strength of AAC
γ_C = partial safety factor for AAC

The design bearing strength of AAC may be determined as follows:

$$f_{ld} = 1.5m\left(\frac{e}{\phi_t}\right)^{\frac{1}{3}} f_{ck} \leq 2.7 f_{cd}$$

where

m = support pressure, taken as $m = 1 + 0.3\frac{n_p}{n_t}$

n_p = number of transverse bars within the transverse pressure
e = distance of axis of transverse bars in the anchorage zone to nearest surface of component.

Determination of the effective length is as follows:

$$t_t = 0.5(t_1 + t_2) \leq 14\phi_t$$

where t_1 and t_2 are distances to the adjacent longitudinal bars as shown below in Figure 2-13.

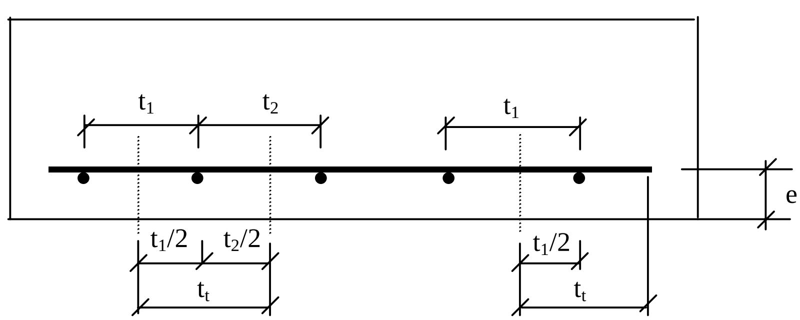

Figure 2-13 Effective Length of Transverse Anchorage Bars

The tensile force acting on the longitudinal reinforcement is determined from structural analysis as follows:

$$F_{ld} = A_s f_{yd} \frac{M_d}{M_{yd}}$$

where

A_s = cross-sectional area of longitudinal bar
M_d = bending moment at critical section
M_{yd} =Design resistant moment of the section, corresponding to f_{yd}

2.5.4 Deflection Limits

PrEN12602 recommends a maximum deflection limit of span/250 for simply supported or cantilevered beams or slabs. For roof panels and walls, the recommended deflection limit is span/200.

For the computation of deflections, PrEN12602 uses the basic bilinear diagram for members subjected to any combination of permanent and variable loads. The bilinear diagram is used to reflect the stiffness of the beam before and after cracking. The method outlined by PrEN12602 for the computation of deflections is equivalent to the method for the computation of deflections outlined in Rilem.

2.5.5 Crack Formation

PrEN12602 addresses crack control by requiring certain minimum amounts of tensile reinforcement.

2.5.6 Bond Strength

PrEN12602 includes an expression to evaluate the bond strength based on the characteristic strength of the material as follows:

$$f_{bd} = 0.8 \frac{f_{bk}}{\gamma_C}$$

where

f_{bd} = design bond strength
f_{bk} = characteristic bond strength determined by testing according to PrEN12269
γ_C = partial safety factor for AAC.

If the characteristic bond strength is not known, bond shall be ignored in the design.

2.5.7 Summary

The draft European Standard PrEN12602 is intended to cover the design of reinforced AAC panels in bending, compression, shear, and torsion, and includes provisions for the evaluation of deflections and recommendations for serviceability conditions, such as deflection limits and cracking limitations. All of the information is presented in SI units. The European Standard PrEN12602 should be used within an ultimate state design approach with checks for the serviceability state of the structural member. PrEN12602 addresses indirectly, bending and combined actions of bending plus axial loads by defining the acceptable strain conditions to be used, leaving the user the option of developing the appropriate equilibrium equations associated with those strain conditions. PrEN12602 does not specify what type of distribution is applicable to the compressive stresses acting on AAC, and it leaves open the possibility of including or ignoring the effect of compression reinforcement for the evaluation of the capacity of the member in bending. However, PrEN12602 requires ignoring the effect of compression reinforcement in members subjected to axial compression or axial compression and bending, unless the compression reinforcement is restrained laterally (by stirrups) to prevent buckling. The European standard PrEN12602 provides the most complete approach for the evaluation of second order effects on AAC members under combined axial loading and bending. The PrEN12602 approach for second order effects includes three different methods as follows:

- A method based on the Euler formula for columns, intended for axially loaded columns only.
- A modified column method intended for the analysis of beam-columns in non-sway structures. This method requires iteration to find the final state of stresses in the structural member.
- An alternative approximate method for the analysis of beam-columns in non-sway structures. This method does not require iteration, but the results are conservative in comparison to the iterative method.

The European Standard PrEN12602 includes design equations for the evaluation of the shear capacity of members with or without shear reinforcement. The shear design equations presented in PrEN12602 consider the possibility of failure because of shear and also the possibility of failure by crushing of AAC in those areas close to the supports where the load is transferred to the supports by direct compression. The European Standard PrEN12602 also provides equations for the structural design of AAC members subjected to a combination of shear and torsion. Those equations consider the case of members with and without shear reinforcement.

The European Standard PrEN12602 presents equations for the evaluation of the anchorage capacity of reinforcement. The equations presented allow for the consideration of bonding between AAC and reinforcement, in addition to the anchorage capacity provided by welded crossbars.

The European Standard PrEN12602 presents equations for the evaluation of the shear capacity of AAC members subjected to punching. This is an important provision for structural members acted by concentrated loads or for stress checks near the supports.

The European Standard PrEN12602 provides serviceability criteria based on stress limitations, crack control, and deflection control. For stress control, PrEN12602 provides the equations for ultimate limit states and the minimum requirements for reinforcement of AAC members, as well as some detailing guidelines. For crack control, PrEN12602 provides general guidelines to control cracking, but not specific crack limits. For deflection control, PrEN12602 provides specific deflection limits and calculation guidelines for the computation of deflections based on the bilinear diagram of moment versus curvature in such a way that the pre and post-cracked stage of the member is considered in the deflection computations. The method outlined for the computation of deflections is essentially identical to the method for deflection computations presented by Rilem.

PrEN12602 mentions fly ash as an acceptable constituent material of AAC but it does not provide any particular design provision for fly ash based AAC reinforced panels.

2.6 CEB Manual of Design and Technology

The CEB manual relies mainly on manufacturer's specifications for AAC, but it does present equations for analysis of bearing area and shear stresses on wall panels used for cladding.

2.6.1 Design Philosophy

The CEB manual uses manufacturer's specifications for design considerations. However, it is stated that design methods based on elastic theory or load factors presented in national codes should be used.

2.6.2 Design Equations

The CEB manual relies on the characteristic strengths given in the manufacturer's specifications; therefore it does not address directly bending, shear, axial force, anchorage deflections and cracking.

2.6.3 Bearing

The CEB manual provides equations to check shear stresses located at the supports for walls. The equations are as follows:

$$\tau_0 = \frac{T}{b_0 \times x_2 \times h_t} \le R'_{bk} \;\; for\; L > 4h_t$$

$$\tau_0 = \frac{T}{b_0 \times x \times h_t} \le R'_{bk} \;\; for\; L < 4h_t$$

Where R'_{bk} is the characteristic compressive strength of aerated concrete, T is the total shear force, b_0 is the width of the wall, and h_t is the total height of the wall.

The concentrated loads should be applied so that the pressure under the load does not exceed $R'_{bk}/3$.

2.6.4 Summary

The CEB manual of design and technology presents structural details and certain design guidelines for reinforced AAC panels. The information is presented in SI units, and the design equations are intended for use within an allowable stress design approach. The equations provided address shear stress checks at supports of horizontal wall slabs and minimum support area to prevent bearing failure. The CEB manual of design and technology does not address fly ash based AAC reinforced panels.

2.7 SIPOREX Technical Manual for the design of vertical and horizontal slabs used for cladding

SIPOREX is an AAC manufacturer that provides information for analysis and design of vertical and horizontal wall panels only.

2.7.1 Design Philosophy

SIPOREX Technical manual uses an allowable stress method with limit states.

2.7.2 Design Equations

SIPOREX Technical manual does not address bending or shear independently; rather, it addresses the interaction between axial load and bending on vertically or horizontally placed walls.

2.7.2.1 Vertical Walls

The actual stresses on vertical walls are obtained by using the principles of strength of materials and considering the different load combinations and load cases acting on the structure.

Given:

$$\lambda = \frac{L}{i} \text{ with } i = \sqrt{\frac{I}{B}}$$

where λ is the slenderness ratio of the slab, I is the moment of inertia, and B is the cross-sectional area of the member.

Also, h is the thickness of the slab, and e is the eccentricity of the resultant due to axial loads prior to the application of the additional eccentricity specified in SIPOREX. If λ is larger than 52e/h or 67e/h, second order effects are considered. However, λ should not exceed 104. The eccentricity and stresses are modified according to the above requirements and are as follows:

- The eccentricity of the resultant axial loads is increased in the critical direction by the larger of 2 cm or L/250.
- The actual stresses are multiplied by the following factors:

$$1 + 0.2 \times \left(\frac{\lambda}{35}\right)^2 \text{ if } \frac{e}{h} \leq 0.75$$

$$1 + 0.15 \times \left(\frac{\lambda}{35}\right)^2 \times \frac{h}{e} \text{ if } \frac{e}{h} > 0.75$$

where the last expression should not exceed 1.4.

2.7.2.2 Horizontal Walls

The actual stresses are obtained by using principles of strength of materials and considering the different load combinations and load cases acting on the structure. SIPOREX Technical manual is based on an allowable stress method and is limited to the following:

For AAC:

- The allowable stresses in compression for AAC to prevent overall buckling are limited to f’c/8.
- The allowable stresses in compression for AAC to prevent local buckling are limited to f’c/8.
- The allowable stresses in compression along the surface of corbels are limited to f’c/6.
- The allowable tangential stresses on corbels are limited to f’c/30.

2.7.3 Summary

SIPOREX Technical manual for the design of vertical and horizontal slabs provides guidelines and design aids for reinforced AAC panels used for cladding. The information is presented in MKS units, and the design equations are intended for use within a limit state design approach. The design equations and design aids provided in the manual cover the determination of the amount of reinforcement for horizontal or vertical walls subjected to transverse loads due to wind. The design equations include provisions for second order effects. The design aids account for the interaction between vertical gravitational loads and lateral wind loads. The information provided does not address specifically the case of fly ash based AAC panels.

2.8 CSR HEBEL Australia Design Sheets, Design References and Formulae

2.8.1 Summary

The CSR HEBEL Australia design sheets are part of a manufacturer’s manual providing design aids and formulas for the computation of stresses on reinforced AAC panels. The formulas are intended for use within an allowable stress design approach. The design aids provide maximum loads as a function of the span for reinforced AAC panels. No reference is made to fly ash based AAC reinforced panels.

3

NUMERICAL RESULTS

3.1 General

This section presents results of design calculations for AAC reinforced elements using the different design methods studied, and comparisons with actual tests performed in the laboratory. It is important to note that not all of the methods cover every design consideration; therefore when a design method is not applicable or it does not cover the specific design consideration under review, it will be clearly marked as N/A.

Tables 3-1 through 3-3 provide details of the AAC reinforced elements (floors, walls, lintel) that were evaluated using the design methods and compared to laboratory test results.

The computations to compare test results with theoretical results were conducted according to the equations described in Chapter 2 of this report.

All numerical calculations are based on the following information:

Table 3-1

AAC Floor Panels

Floor Panel	AAC Strength (Mpa)	Width (in)	Thickness (in)	Span (ft)	Bottom Longitudinal Reinforcement (60 ksi)	Top Longitudinal Reinforcement (60 ksi)	Test Failure Type
1	5	24	8	13.67	6-7mm	2-7mm	Shear
2	5	24	8	16.50	4-7mm	4-7mm	Tensile
3	5	24	8	16.50	6-7mm	4-7mm	Compressive
4	5	24	6	12.50	10-7mm	8-7mm	Shear

Table 3-2

AAC Wall Panels

Wall Panel	AAC Strength (Mpa)	Width (in)	Thickness (in)	Span (ft)	Longitudinal Reinforcement (60 ksi)
1	5	24	6	8	10-7mm
2	5	24	8	8	10-7mm
3	5	24	9.5	8	10-7mm

Table 3-3

AAC Lintel

	AAC Strength (Mpa)	Width (in)	Thickness (in)	Span (ft)	Bottom Longitudinal Reinforcement (60 ksi)	Top Longitudinal Reinforcement (60 ksi)	Test Failure Type
Lintel	5	24	8	18	6-7mm	6-7mm	Tensile

3.2 Results

3.2.1 Moment Capacity of Floor Panels

Table 3-4 contains moment capacities for the floor panels listed in Table 3-1. Information regarding the calculation of moment capacities was available in Rilem and DIN 4223 only. In the calculations, all of the safety factors were removed in order to represent the ultimate capacity that is compared to the test results. Laboratory test values were only available for Panels 2 and 3, as Panels 1 and 4 failed in shear.

Table 3-4

Moment Capacity of Floor Panels (kip-ft)

Floor Panel	Test Value	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
1	* N/A	10.95	10.13	N/A	N/A	N/A	N/A	N/A
2	10.30	9.90	6.94	N/A	N/A	N/A	N/A	N/A
3	13.50	14.90	10.13	N/A	N/A	N/A	N/A	N/A
4	* N/A	12.46	7.05	N/A	N/A	N/A	N/A	N/A

*N/A - Not available because specimen failed in shear

N/A - Not applicable because method does not contain any information

3.2.2 Moment Capacity of Lintel

Table 3-5 contains the moment capacities for the lintel in Table 3-3. Again, Rilem and DIN 4223 were the only available design methods with the applicable equations. In the calculations, all of the safety factors were removed in order to represent the ultimate capacity that is compared to the test results.

Table 3-5

Moment Capacity of Lintel with Multi-Layer Reinforcement (kip-ft)

Test Value	*Rilem	*DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
38.04	29.68	32.14	N/A	N/A	N/A	N/A	N/A

* Modified methods to account for multi-layer reinforcement

N/A – Not applicable because method does not contain any information

3.2.3 Shear Capacity of Floor Panels

Table 3-6 lists the shear capacities for the floor panels in Table 3-1. The design methods containing information used for calculations were Rilem, DIN 4223, and PrEN 12602. In the calculations, all of the safety factors were removed in order to represent the ultimate capacity that is compared to the test results. Test values were only available for Panels 1 and 4 due to bending failures occurring in Panels 2 and 3.

Table 3-6

Shear Capacity of Floor Panels without Shear Reinforcement (kips)

Floor Panel	Test Value	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
1	3.75	3.94	4.00	N/A	4.72	N/A	N/A	N/A
2	* N/A	3.54	4.10	N/A	4.23	N/A	N/A	N/A
3	* N/A	3.82	4.00	N/A	4.72	N/A	N/A	N/A
4	3.30	3.56	2.70	N/A	5.12	N/A	N/A	N/A

* N/A – Not available because specimen failed in bending or compression

N/A – Not applicable because method does not contain any information

3.2.4 Shear Capacity of Lintel

Table 3-7 contains the shear capacities for the lintel in Table 3-3. In this case, Rilem and PrEN 12602 were the only available design methods with provisions for shear reinforcement in the design equations. In the calculations, all of the safety factors were removed in order to represent the ultimate capacity that is compared to the test results.

Table 3-7

Shear Capacity of Lintel with Shear Reinforcement (kips)

Test Value	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
8.10	8.68	N/A	N/A	18.36	N/A	N/A	N/A

N/A – Not applicable because method does not contain any information

3.2.5 Number of Transverse Bars for Anchorage in Floor Panels

Table 3-8 contains the number of transverse bars in the floor panels from table 3-1. Rilem, DIN 4223, and PrEN 12602 were the only available design methods that contained information on anchorage bar calculations.

Table 3-8

Number of Transverse Bars in Floor Panels

Floor Panel	Actual Number of Bars	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
1	8	4	8	N/A	3	N/A	N/A	N/A
2	8	3	7	N/A	2	N/A	N/A	N/A
3	8	4	7	N/A	3	N/A	N/A	N/A
4	8	3	7	N/A	5	N/A	N/A	N/A

N/A – Not applicable because method does not contain any information

3.2.6 Deflections of Floor Panels

Tables 3-9 and 3-10 provide the deflections at cracking and the maximum deflections calculated for the floor panels listed in Table 3-1. Rilem, DIN 4223, and PrEN 12602 have design information for computing deflections at cracking and maximum deflections.

Table 3-9

Deflections at Cracking of Floor Panels (mm)

Floor Panel	Test Value	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
1	7.5	7.3	23.0	N/A	7.3	N/A	N/A	N/A
2	7.0	6.5	25.2	N/A	6.5	N/A	N/A	N/A
3	9.0	8.7	32.0	N/A	8.7	N/A	N/A	N/A
4	14.0	10.8	51.3	N/A	10.8	N/A	N/A	N/A

N/A – Not applicable because method does not contain any information

Table 3-10

Maximum Deflections of Floor Panels (mm)

Floor Panel	Test Value	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
1	19.8	12.6	98.8	N/A	12.6	N/A	N/A	N/A
2	74.4	30.7	95.8	N/A	30.7	N/A	N/A	N/A
3	42.4	22.8	118.6	N/A	22.8	N/A	N/A	N/A
4	52.0	29.1	120.7	N/A	29.1	N/A	N/A	N/A

N/A – Not applicable because method does not contain any information

3.2.7 Axial Capacity of Wall Panels

Table 3-11 shows the axial capacities of the wall panels listed in Table 3-2. The design methods containing information used for calculations were Rilem and DS 420, which use the same equations, and PrEN 12602. In the calculations, all of the safety factors were removed in order to represent the ultimate capacity that is compared to the test results.

Table 3-11

Axial Capacity of Wall Panels (kips)

Wall Panel	Test Value	Rilem	DIN 4223	DS 420	PrEN 12602	CEB	SIPOREX	CSR HEBEL Australia
1	65.72	65.76	N/A	65.76	18.82	N/A	N/A	N/A
2	74.19	87.69	N/A	87.69	33.29	N/A	N/A	N/A
3	82.25	104.13	N/A	104.13	56.98	N/A	N/A	N/A

N/A – Not applicable because the method does not contain any information

Note: Rilem and DS 420 use the same formulas for axial computations

4

DISCUSSION OF RESULTS

4.1 General

This chapter compares among the various design parameters as computed using the different methods and measured by physical testing in the laboratory. The results from Chapter 3 are represented graphically and significant differences are identified and discussed. The discussion section following each graph identifies specific trends and differences in the design methods and test values.

4.2 Comparison of Design Calculations with Test Results

This section provides graphical comparisons of results obtained from the design calculations and those obtained from testing.

4.2.1 Analysis of Bending Capacity (Floor Panels)

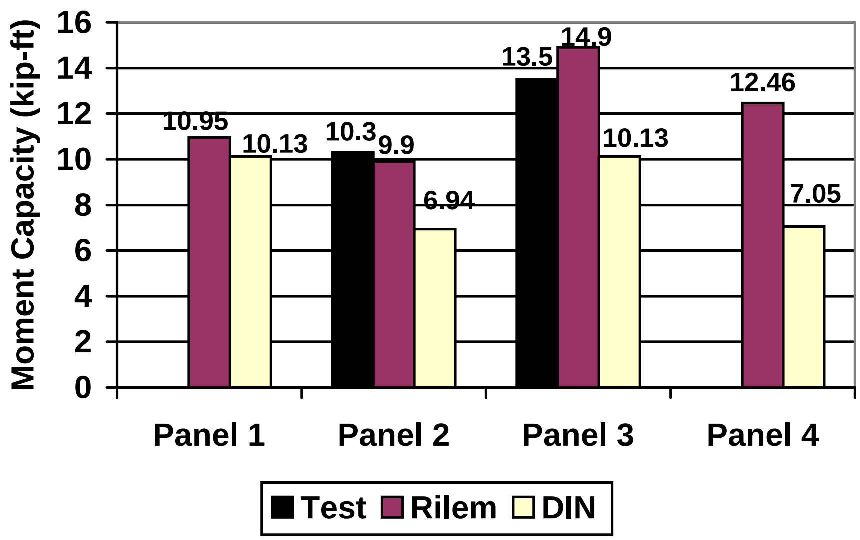

Figure 4-1 Comparison of Moment Capacity for Floor Panels

Figure 4-1 shows the differences in moment capacity between Rilem and DIN 4223. The test values were only available for Panels 2 and 3 due to shear failures in Panels 1 and 4. It appears that Rilem does provide close estimates compared to the test results; however, this cannot be generalized since only limited test data was available for comparison. When comparing the differences between the design methods, there is an inconsistent pattern with percent differences ranging from 7.5% in Panel 1 to 43% in Panel 4. When comparing these methods to the test values, Rilem is closer than DIN 4223 in both cases with percent differences of 4% in Panel 2 and 9.5% in Panel 3. Overall, there is no agreement between Rilem and DIN 4223.

4.2.2 Analysis of Bending Capacity (Lintel)

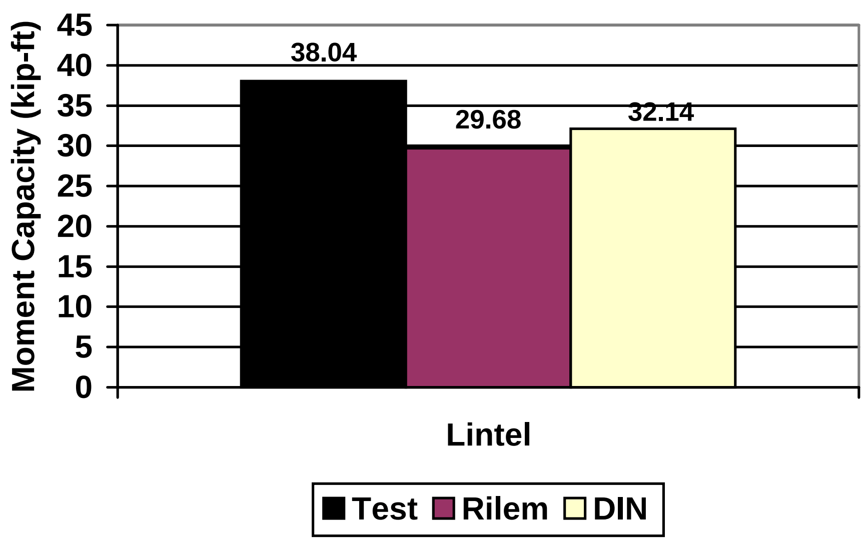

Figure 4-2 Comparison of Moment Capacity for Lintel

Figure 4-2 shows the differences in moment capacity between Rilem and DIN 4223 for one lintel with multi-layer reinforcement. When comparing Rilem and DIN 4223 to the test value, the differences are 22% and 16% in the conservative side. These differences could be due to the analytical formulations not accounting for more than one layer of steel reinforcement.

4.2.3 Analysis of Shear Capacity (Floor Panels)

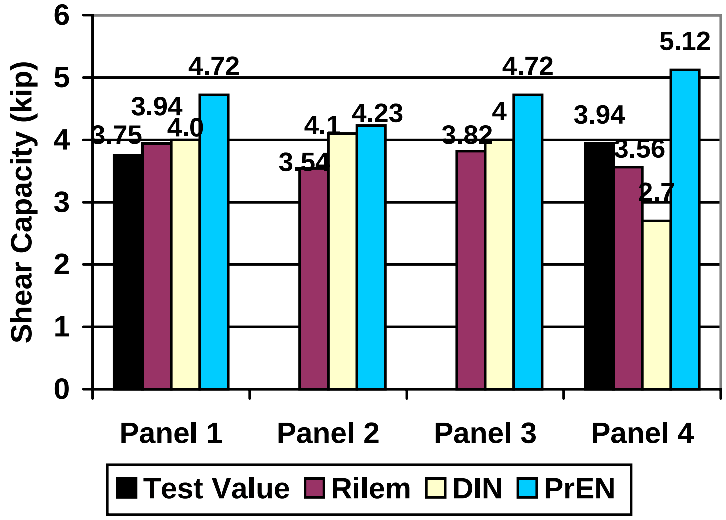

Figure 4-3 Comparison of Shear Capacity for Floor Panels

Figure 4-3 shows the differences in shear capacity between Rilem, DIN 4223, and PrEN 12602 for floor panels without shear reinforcement. The test values were only available for Panels 1 and 4 due to tensile and compression failures in Panels 2 and 3. It appears that Rilem provides close estimates (5%) to test values. PrEN 12602 overestimates shear as compared to the test results (20% to 23%) and the other analytical methods, with the highest percent difference at 47 %. There are mixed results, with theoretical predictions being non-conservative in one case (Panel 1) and conservative in another case (Panel 4), except for PrEN 12602, which is non-conservative in either case.

4.2.4 Analysis of Shear Capacity (Lintel)

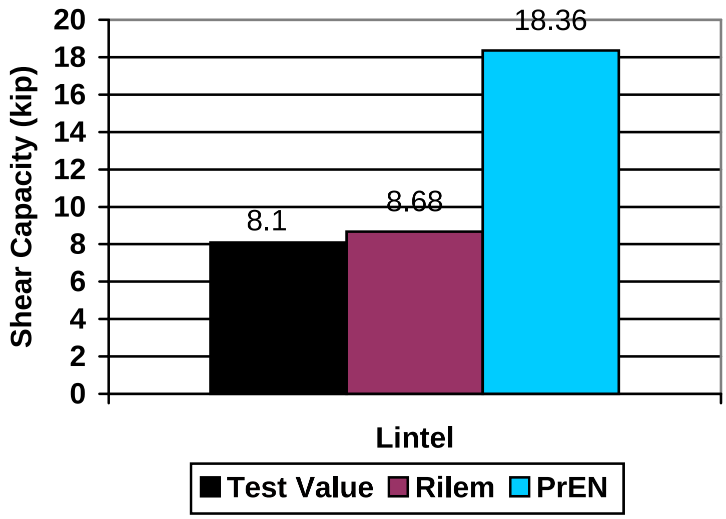

Figure 4-4 Comparison of Shear Capacity for Lintel

Figure 4-4 shows the differences in shear capacity between Rilem and PrEN 12602 for one lintel with shear reinforcement. It appears that Rilem values are close estimates to the test values with a percent difference of only 7%. PrEN 12602 is very non-conservative with a difference of 56 %.

4.2.5 Analysis of Number of Transverse Anchorage Bars (Floor Panels)

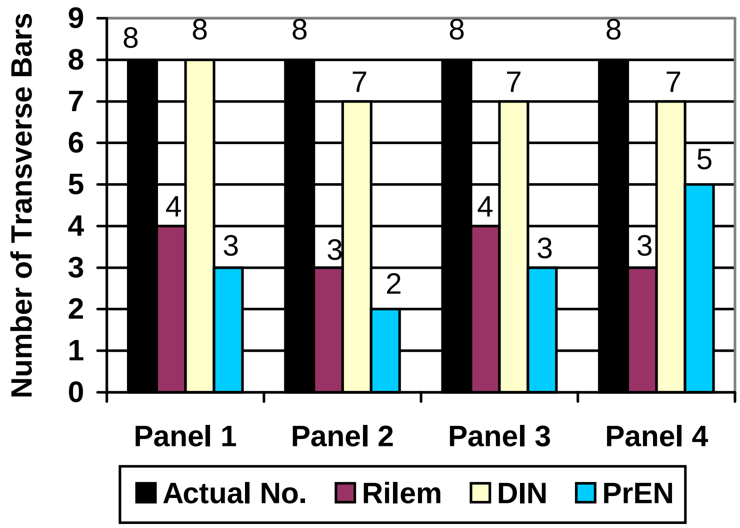

Figure 4-5 Comparison of Number of Transverse Anchorage Bars for Floor Panels

Figure 4-5 shows the differences in the number of transverse bars for each floor panel using Rilem, DIN 4223, and PrEN 12602. The actual number of transverse bars for all of the panels is 8. DIN 4223 compares the best with 8 bars in Panel 1 and 7 in Panels 2, 3, and 4. Rilem consistently produces results from 3 to 4 bars. The PrEN method produced values from 2 bars in Panel 2 and 5 bars in Panel 4. Although DIN 4223 produced results that compared well with the actual test panels, additional testing is needed to study the effects of using less number of cross bars. It should be pointed out that the test panels did not fail due to anchorage, therefore the number of bars needed for anchorage is not known.

4.2.6 Analysis of Deflections (Floor Panels)

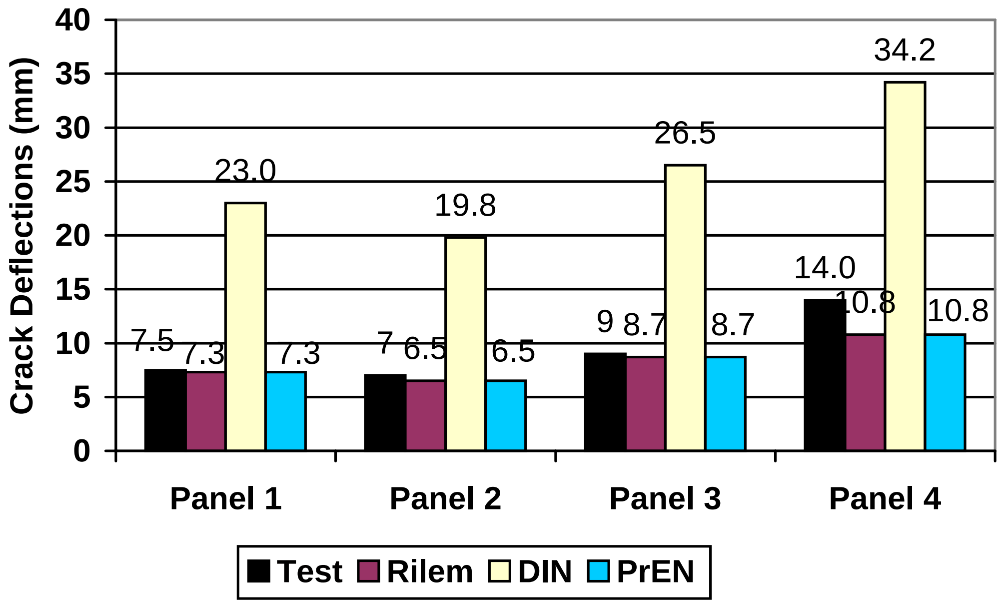

Figure 4-6 Comparison of Deflections at Cracking for Floor Panels

Figure 4-6 shows the differences in deflections at cracking of the floor panels for Rilem and DIN 4223. The capacities for Rilem and PrEN 12602 are the same in every panel because these methods are equivalent. It appears that Rilem and PrEN 12602 provide good estimates of deflections at cracking when compared to the test deflections with a maximum percent difference of 23% in Panel 4. However, DIN 4223 overestimates deflections at cracking, which causes large percent differences when compared to the test values and to Rilem and PrEN 12602.

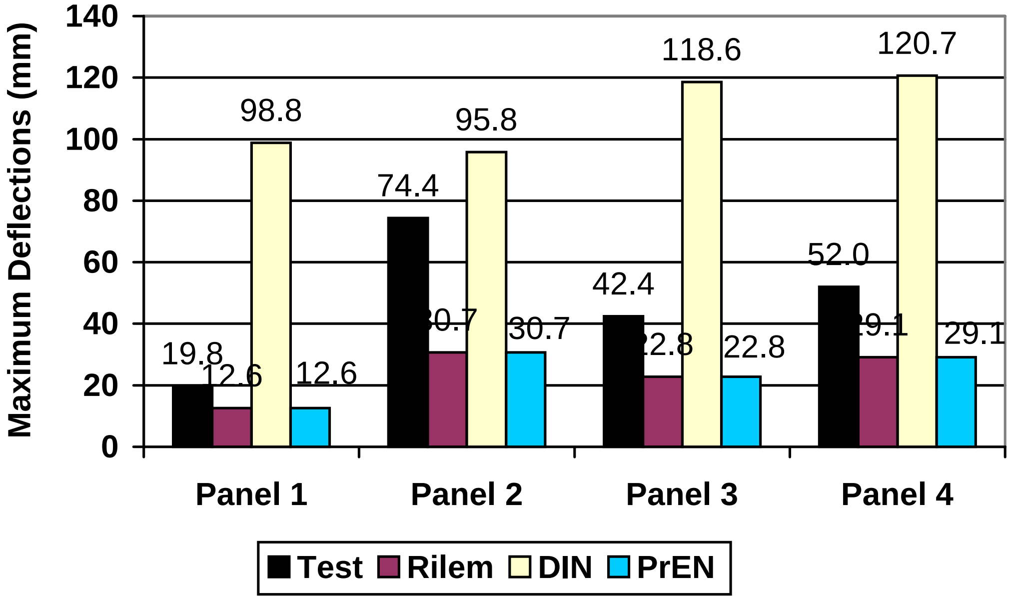

Figure 4-7 Comparison of Maximum Deflections for Floor Panels

Figure 4-7 shows the differences in the maximum-recorded deflections for each floor panel according to Rilem, DIN 4223, and PrEN 12602. Again, the capacities for Rilem and PrEN 12602 are the same in every panel because these methods are equivalent. These results show the greatest differences than any other design parameter and it appears that no method accurately predicts deflections. However, this is not uncommon for deflections calculations. The Rilem and PrEN values are obviously closer to the test values with percent differences ranging from 36% in Panel 1 to 59% in Panel 2. The DIN 4223 method for computing deflections does not consider cracking effects. This greatly affects the results and causes huge percent differences in the methods and the test results. The percent differences between DIN 4223 and the test values range from 22% in Panel 2 to 80% in Panel 1.

4.2.7 Analysis of Axial Capacity (Wall Panels)

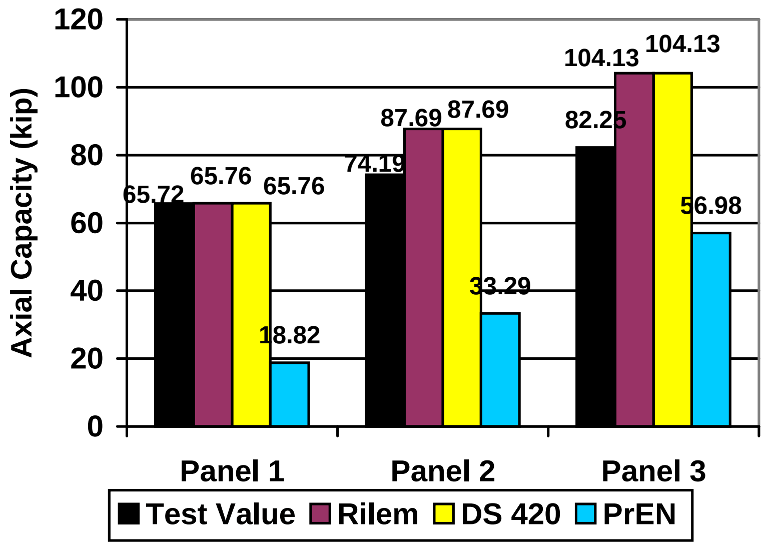

Figure 4-8 Comparison of Axial Capacity for Wall Panels

Figure 4-8 shows the differences in the axial capacity between Rilem, DS 420, and PrEN 12602. The capacities for Rilem and DS 420 are the same in every panel because these methods are equivalent. Only Panel 1 had analytical predictions that closely compared with the test value. For Panels 2 and 3 the analytical methods tend to over estimate the panel capacity. PrEN 12602 is very conservative, producing extremely low results in comparison to the other methods and to the test values, with the highest percent difference at 71%. The Rilem and DS 420 methods are fairly consistent with the test results with percent differences ranging from 0.1 to 21%. Also, the Rilem and DS 420 capacities are consistently larger than the test values, which demonstrate that these methods are non-conservative.

5

CRITIQUE OF DESIGN METHODS

5.1 General

Chapter 5 contains a general critique of each design method based on the observations made in Chapter 2 and the results and comparisons made from Chapters 3 and 4. The following sections discuss the contents of every design method analyzed, identifying the strengths and weaknesses of each.

5.2 Rilem Recommended Practice

Rilem recommended practice includes a relatively complete set of design equations for bending, shear, axial force, anchorage, and bearing as well as serviceability checks such as deflections and cracking. The design equations are presented in SI units and are intended to be used with an ultimate state design approach. The equations to evaluate the capacity of members in bending cover a wide range of design situations from under-reinforced members to over-reinforced members. However these equations are not very accurate in predicting the type of failure of the bending member. Also, the bending equations are not applicable for lintels with multi-layer reinforcement. In that case, it is up to the user to develop the appropriate bending equations based on compatibility of strains and equilibrium of forces acting at the cross section. The shear equations for members without shear reinforcement are accurate for typical AAC strengths (AAC 5.0) and standard levels of reinforcement. For non-typical situations, the shear equations provided are not accurate in predicting the shear strength of members without shear reinforcement. For members with shear reinforcement, the equations provided are not accurate to predict the shear strength of the examples analyzed. The equations for axial force are a subset of those presented in Danish Standard DS420. Those equations do not include considerations of second order effects, nor the effect of the longitudinal reinforcement. The method described in Rilem for the analysis of bearing stresses is cumbersome and difficult to use. The method outlined for the computations of deflections is very complex and the results are not accurate when compared to testing measurements. The procedure used for the evaluation of the anchorage strength and the determination of the required number of anchorage bars for the longitudinal steel is relatively easy to follow for simple loading cases, such as simply supported slabs with uniformly distributed loads. However, the applicability of the anchorage equations becomes extremely complex for cases with continuous members, with intermediate supports, with distributed loads other than uniform loads, or with members with several layers of reinforcement, such as lintels. Rilem also presents minimum requirements to control cracking. Those requirements are very stringent and of limited applicability in real designs. Finally, it is necessary to point out that some of the design equations need clarification of terminology and notation, since some symbols are used repetitively implying different meaning in every case. Rilem does not address fly ash based AAC reinforced panels.

5.3 DIN 4223 Guidelines for Reinforced AAC Roof and Ceiling Slabs

DIN 4223 is limited to bending members and considers only two types of AAC (GSB 35 and GSB 50). The original units are Meter-Kilogram-Second (MKS), but standardization is needed because the equations use meters and centimeters simultaneously. The equations for bending are easy to use, but they do not consider the effect of compressive reinforcement. Also, the bending equations do not include provisions for multi-layer reinforced members, such as lintels. The shear equations are limited to members without shear reinforcement. The deflection provisions do not consider cracking effects or compressive reinforcement, since they are intended for serviceability conditions only. This makes the equations for deflections extremely unreliable and causes the cracking and maximum deflections to be extremely conservative. This is shown in Figure 4-6 and Figure 4-7. The anchorage equations are not easily applicable to multi-layer reinforced members, such as lintels. DIN 4223 does not address fly ash based AAC reinforced panels.

5.4 Danish Standard DS 420 Code of Practice for the Structural Use of Lightweight Concrete

The Danish Standard DS 420 focuses mostly on vertical walls. No consideration is given for different types of failure, such as ductile or brittle failure, and no design information is given for members in bending only. The shear information provided is only applicable to axially loaded members acted by shear forces on their own plane. This information is useful for the evaluation of shear strength of reinforced walls, but is not directly applicable to slabs or lintels. The cracking limits are too stringent for practical use. The design equations provided for the check of bearing stresses at supports are similar to those found in Rilem and their use is cumbersome and non-conservative. The units used are not clarified in the design equations. DS 420 does not address fly ash based AAC reinforced panels.

5.5 Draft European Standard PrEN12602, "Prefabricated Reinforced Components of Autoclaved Aerated Concrete"

The European Standard PrEN12602 is by far the most complete document in terms of design methods for reinforced AAC panels. However, it lacks clear explanations of how to evaluate the bending capacity of AAC members with no explicit equations provided. PrEN 12602 contains a complete set of equations for the calculation of shear capacities with and without shear reinforcement. The equations for the axial capacity of walls are overly conservative and require modifications. PrEN 12602 does contain information on buckling and second order effects, but the evaluation of second order effects may be cumbersome in some instances. Anchorage and bonding calculations are very straightforward and easily followed, but their results are non-conservative. This method presents a method for calculating deflections that is identical to the Rilem method, but PrEN 12602 provides a more consistent approach in terms of notation. PrEN 12602 contains reasonable deflection limits, which are much less stringent than Rilem. PrEN 12602 contains a serviceability limit state for crack control. Cracking is limited by specifying minimum and maximum spacing for reinforcing bars and minimum and maximum diameters for reinforcing bars. This standard is the only one considering the effects of punching and torsion. PrEN 12602 does not address fly ash based AAC reinforced panels, although it allows its use without any particular restrictions.

5.6 CEB Manual of Design and Technology

The CEB manual of design and technology provides design equations for wall members only. The equation variables are not clearly defined and units are not specified in design equations. The bearing capacity equations do not take into account the trapezoidal stress distribution at the support. The CEB manual does not address fly ash based AAC reinforced panels.

5.7 SIPOREX Technical Manual for the design of vertical and horizontal slabs used for cladding

SIPOREX technical manual has limited use since it is intended for vertical and horizontal slabs used for cladding. The manual does not have clear explanations about the units used in the design equations and design aids. The design equations and design aids only cover the interaction of axial loads and bending, considering longitudinal reinforcement and second order effects. Other important issues such as shear, bearing, deflections, and cracking are not addressed. The SIPOREX technical manual does not address fly ash based AAC reinforced panels.

5.8 CSR HEBEL Australia Design Sheets, Design References and Formulae

CSR HEBEL Australia Design Sheets, Design References and Formulae addresses reinforced panels in bending with limited information about design methods or design philosophies. The design aids provided are intended as a guide for determination of the proper type of AAC member in a situation where the span and superimposed load are known. This document does not address axially loaded members or serviceability conditions. The CSR HEBEL Australia Design Sheets do not address fly ash based AAC reinforced panels.

6

CONCLUSIONS

Based on the literature review and the analytical work performed, the following conclusions may be drawn:

1) The literature review produced little information on fly ash based AAC.
2) The literature review identified seven documents containing relevant information for the design of reinforced AAC panels. These documents are listed in items 3 through 19 below.
3) Rilem provides a very detailed analysis on design of AAC panels including bending, shear, axial capacity, deflections, anchorage and bearing capacity.
4) Rilem uses an ultimate strength design method, which is based on a semi-probabilistic approach.
5) Some of the equations presented in Rilem need clarification of terminology and notation.
6) DIN 4223 presents provisions for reinforced AAC panels used as slabs and includes such topics as, bending, shear, deflections, anchorage, and crack formation.
7) DIN 4223 uses an allowable stress design approach.
8) DIN 4223 is limited to bending members and considers only two types of AAC (GSB 35 and GSB 50).
9) DS 420 presents methodologies for analysis of shear, axial capacity (which is equivalent to Rilem), bearing, and crack formation.
10) DS 420 uses an ultimate strength design approach and focuses mainly on vertical walls.
11) PrEN 12602 is a very detailed document covering many aspects of AAC. It covers bending, shear, axial capacity, anchorage, bond strength, deflections (which is equivalent to Rilem), crack formation, and torsion.
12) PrEN 12602 presents provisions for ultimate and serviceability limit states.
13) PrEN 12602 lacks explicit equations regarding the bending capacity of reinforced AAC panels.
14) The CEB manual mainly relies on manufacturer's specifications for design considerations, but provides some guidelines for analyzing bearing and shear stresses in wall panels.
15) The equation variables in the CEB manual are not clearly defined and units are not specified in the design equations.
16) SIPOREX is an AAC manufacturer that provides a manual with product information regarding vertical and horizontal wall panels.

17) SIPOREX does not present clear explanations about the units used in the design equations and the design aids.
18) The CSR HEBEL design sheets are part of a manufacturer's product manual, which provides design aids for evaluating stresses in reinforced AAC panels.
19) The CSR HEBEL design sheets contain limited information regarding design methods or design philosophies.
20) In predicting the bending strength, Rilem provided better estimates than the other design methods.
21) Shear strength was better estimated by Rilem's procedures as compared to PrEN 12602 or DIN 4223.
22) The number of crossbars for anchorage in the test panels closely matched the values computed by DIN 4223. However, the actual number of needed crossbars for anchorage could not be determined.
23) None of the existing methods predicted deflections very well, but this is not uncommon for deflection computations. Rilem provided the closest prediction as compared to the test results.
24) When analyzing axial capacity, Rilem (or DS 420) estimated the actual capacity better than the other design methods available. PrEN 12602 was extremely conservative in predicting the axial capacity.

7

RECOMMENDATIONS

Based on the review of different methodologies, it is clear that an American AAC methodology should address at least the following design topics:

- Bending strength of members with single or multiple layers of reinforcement
- Shear strength of bending members with or without shear reinforcement
- Axial strength of members with or without eccentricity and including considerations for buckling and second order effects.
- Shear strength of axially loaded members
- Anchorage of reinforcement
- Bonding
- Bearing strength
- Punching strength

Also the following serviceability conditions should be addressed:

- Short and long term deflections
- Deflection limits
- Cracking load and cracking moment
- Crack control

The development of a completely new U.S. based design methodology for reinforced AAC that covers the design aspects previously enumerated may require extensive testing and calibration and important resources in terms of money and time. The time required for the development of such a design methodology may affect negatively the growth and expansion of the AAC market. In order to address those issues, a two-step research program based on the current state of knowledge is recommended for the development of a U.S. methodology for the design of reinforced AAC panels.

Phase I requires the adoption of some of the methodologies reviewed in the report with some modifications in order to present those methodologies in a form that is acceptable and common to U.S. engineers. Phase I will be presented as a set of design guidelines, and it will include provisions for fly ash based reinforced AAC with somewhat higher safety factors due to the lack of information about the structural behavior of this material. Table 7-1 shows the different methods that would be used in Phase I to build the U.S. design methodology for the design of reinforced AAC members.

Table 7-1

Design Methods for the Proposed U.S. Design Methodology for Phase I

Topic	Existing Method to be Referenced	Notes
Bending strength	Rilem	Calibration required to bring results to conservative side.
Shear strength of bending members	Rilem	Complete set of equations to cover members with and without shear reinforcement. Requires calibration.
Axial strength	DS420 (Rilem)/PrEN12602	DS420 is adequate for the evaluation of the axial strength; PrEN12602 provides the considerations for buckling and second order effects.
Shear strength of axially loaded members	DS420	DS420 is the only method with information available on this topic.
Torsion strength	PrEN12602	PrEN12602 is the only method with information available on this topic.
Anchorage of reinforcement	PrEN12602	Straightforward procedure. Requires calibration to accurately model the reinforcement behavior.
Bonding	PrEN12602	Straightforward procedure. Requires calibration.
Bearing strength	DS420	Requires clarification of procedure.
Punching strength	PrEN12602	Information provided is easily applicable. Calibration required by means of testing.
Deflection computations	PrEN12602/Rilem	Identical methods. Require calibration to match test results. PrEN12602 presents a more consistent approach in terms of notation.
Deflection limits	PrEN12602	Less stringent requirements in comparison to Rilem.
Crack control	PrEN12602	Crack control provided by specifying minimum and maximum spacing of reinforcing bars and by specifying minimum and maximum diameter of reinforcing bars.

The implementation of Phase I should ultimately lead to Phase II, which will involve a testing program that will verify the existing design methods developed in Phase I. Special emphasis during Phase II will be given to the development of design equations for fly ash based AAC. Additionally, Phase II will provide the design formulations in a specification/commentary format using mandatory language.

8

RECOMMENDATIONS FOR FUTURE RESEARCH

The objective of the work proposed herein is to develop methodologies for the design of AAC panels based on U.S. practice. This work will consist of a two-phase research program.

In Phase I, the existing knowledge and determined information presented in this report will be modified and adapted for use by engineers in the U.S. The design methodology will include all of the required design topics listed in Chapter 7. Changes will include the review of safety factors, notation used, and units of measure. The developed recommended design procedures will be user friendly and easy to follow by the practicing engineer and will include a commentary to explain the basis for some of the provisions. In addition, detailed design examples illustrating the use of the methods developed will be included. Design aids will also be developed to assist the engineer during the design process to simplify the overall design of AAC.

In Phase II, an experimental program will be carried out to verify the design procedures established in Phase I, and to shed light on the behavior of fly ash based AAC through physical testing of AAC components. As part of Phase II, the design methods will be revised and presented in mandatory specification language with an adjoining commentary that will assist in explaining various provisions of the specification.

9

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