AN INTRODUCTION TO AUTOCLAVED AERATED CONCRETE INCLUDING DESIGN REQUIREMENTS USING STRENGTH DESIGN

by

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Abstract

Autoclaved Aerated Concrete (AAC) is a lightweight concrete building material cut into masonry blocks or formed larger planks and panels. Currently it has not seen widespread use in the United States. However, in other parts of the world it use has been used successfully as a building material for over fifty years. AAC is a relatively new (at least to the United States) concrete masonry material that is lightweight, easy to construct, and economical to transport. Its light weight is accomplished through the use of evenly distributed microscopic air bubbles throughout the material; these bubbles result in a lightweight concrete that is composed of a latticework around spherical voids. This report details the history, physical properties, manufacturing process, and structural design of AAC. This report includes an explanation of the 2005 Masonry Standards Joint Committee (MSJC) Code for the design of AAC members subjected to axial compressive loads, bending, combined axial and bending, and shear. An example building design using AAC structural components is provided. This report concludes that AAC has important advantages as a structural building material that deserves further consideration for use in the United States.

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Dedication

For Them.

CHAPTER 1 - Introduction

Autoclaved aerated concrete (AAC) is a lightweight concrete material that was developed in Sweden approximately 85 years ago but only recently, as early as 1990 in the Southeast, has it been used or produced in the United States (www.gostructural.com). It is a lightweight building material that is easy to build with, has great thermal properties, and can be easily produced from locally available materials. AAC is commonly found as masonry block units or as larger planks that can be used as wall components or as roof or floor components (Figure 1.1). AAC has a high percentage of air making up its volume and the materials that are used to make it can be recycled from waste AAC material. Recycled AAC can be ground up finely and can be used as the aggregate in the new mixture. Also, the energy that is required to produce AAC is much lower than other masonry products (www.eaaca.org).

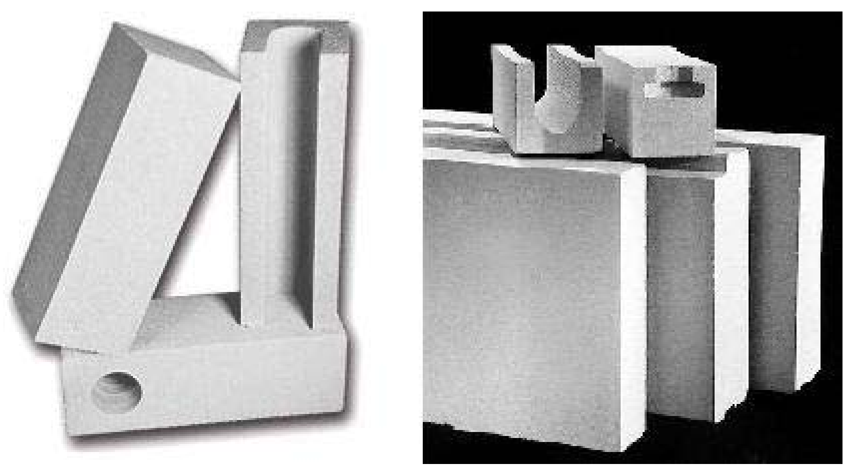


Figure 1.1: AAC Masonry Block and Plank/Panels
(left: www.e-crete.com, right: www.masonryinnovations.net)

This report provides detailed information on the history, mechanical properties, a description of the manufacturing process, and the structural design requirements of autoclaved aerated concrete. For the structural design requirements, the various strength requirements for axial, bending, and shear, are explained and examples are provided. Using the 2005 Masonry

Standards Joint Committee Code (2005 MSJC Code), design of a simple building is provided using AAC structural components.

The purpose of this paper is only to inform the reader of the capabilities of autoclaved concrete as well as provide examples on design approach as set forth by the 2005 MSJC Code. It is not meant to create a new design approach nor is it to provide newly proposed analytical provisions.

CHAPTER 2 - History

Compared to the twelve thousand plus years of masonry history, (masonry was used in the Egyptian pyramids, Mayan civilization, etc) the history of autoclaved aerated concrete begins much, much later on in the masonry history timeline. The earliest specification, developed by the American Society for Testing and Materials International (ASTM International), for the design of AAC was released just nine years ago in 1998 and was a specification covering the structural design of non-load bearing and bearing walls of AAC. Although the idea of aerating concrete to make it lighter is not a new idea the idea of autoclaved aerated concrete was first developed and patented in the early nineteen twenties in Sweden. A Swedish architectural science lecturer, by the name of Johan Axel Eriksson, first discovered AAC in 1923 almost accidentally while working on some aerated concrete samples he placed them in an autoclave to speed the curing process (www.cfg.co.nz). Its application was similar to masonry but it was more lightweight. The use of AAC spread through Europe, then Asia, then Australia, and has just only recently (recently being the early 1990's) come to United States. AAC started out in the American southeast and has slowly been spreading in its use to other parts of the country. In 1998 the Autoclaved Aerated Concrete Products Association (AACPA) was formed to promote the use of autoclaved aerated concrete in the United States (www.aacpa.org). The AACPA is similar to the European Autoclaved Aerated Concrete Association (EAACA) which was created in 1988.

Currently, in the United States, there are two producers of autoclaved aerated concrete. Xella Aircrete North America Inc. (Hebel) has plants located in Texas, Georgia, and Mexico as well, and AERCON is located in Florida (www.aacpa.org). The annual production of AAC in the United States is not currently available, however, the annual production capacity of the largest North American producer of AAC (Hebel's Georgia Facility) can produce approximately 2.7 billion cubic feet (250,000 cubic meters) per year (www.xella-usa.com)

CHAPTER 3 - Manufacturing Process

The production of Autoclaved Aerated Concrete (AAC) is similar in nature to the production of clay masonry units or even precast concrete. The materials used in AAC are similar to the concrete normally used in structural components. The manufacturing process of AAC can be likened to the process of baking bread, and can be summarized into five main steps:

1) Assembling and mixing of the raw materials.
2) Adding of the expansion agent.
3) Expansion, shaping, pre-curing., and cutting.
4) Final curing utilizing an autoclave.
5) Packaging and shipping.

The image below depicts the manufacturing process beginning with the mixing of raw materials and ending with the shipping stage.

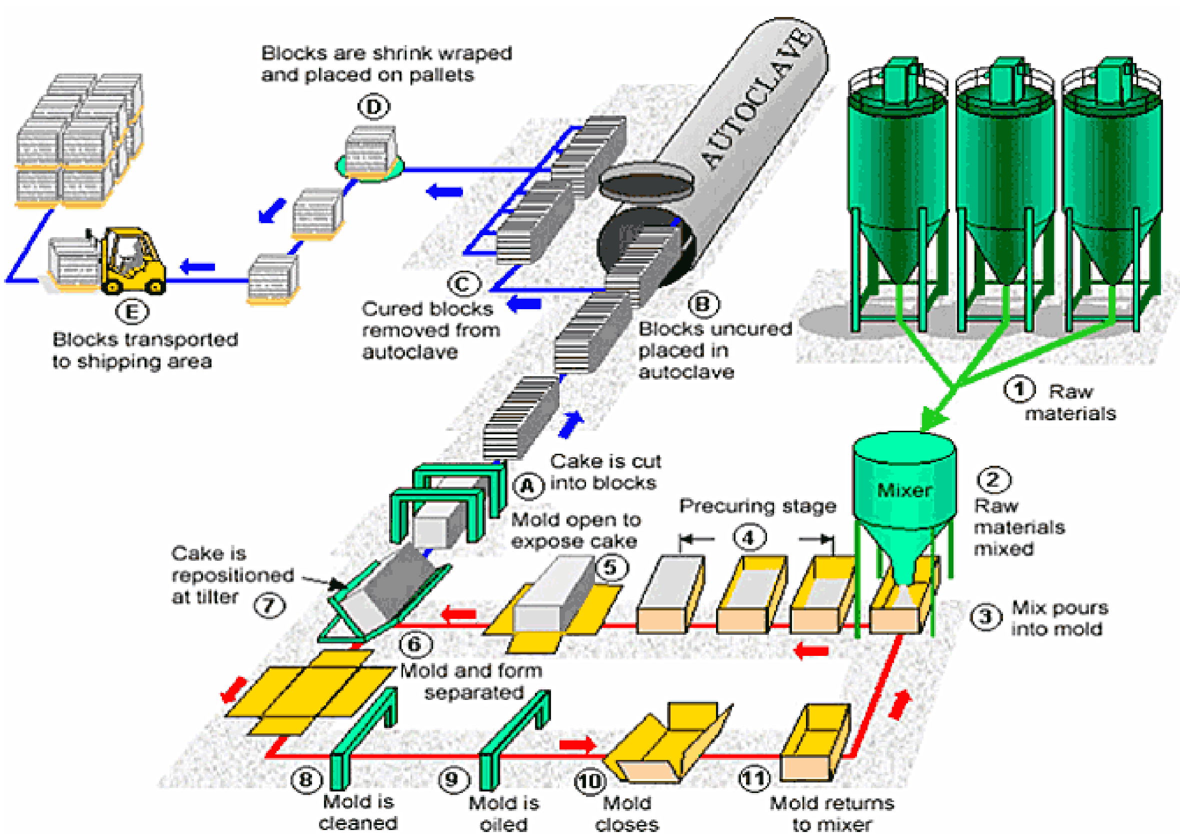


Figure 3.1: Manufacturing Process of AAC Masonry Units (www.aacstructures.com)

Step 1: Assembling and mixing of the raw materials

The production of AAC starts with the raw materials of silica, cement, lime, and water. The silica, which is used for the aggregate, is made from finely ground quartz. Fine sand can be used in place of silica. Also, fly ash, slag, or mine tailings which are the ground up remains from mining operations, can be used as aggregate in combination with the silica. These materials are the fine aggregate of the concrete mix. The aggregate needs to be a fine gradation, not course or large material because a larger aggregate interferes with the internal structure created by the microscopic bubbles produced in step 2. Portland cement is used, just as it is used in normal concrete mixes. Portland cement is the binding agent which holds the aggregate together. It reacts with water in a process called hydration and then hardens, bonding all the aggregates together to form a solid material. All these mixed together with water form the base AAC mixture. The raw components are then mixed together with water in a large container.

Step 2: Adding of the expansion agent

In making a loaf of bread, yeast is added to the dough mixture to make the bread rise. In a similar way, an expansion agent is added to the concrete mix to increase its volume. Yeast produces carbon dioxide which causes the dough to expand. In autoclaved aerated concrete, the expansion agent that is used is aluminum powder or paste. The aluminum reacts with the calcium hydroxide and water in the mixture creating millions of tiny hydrogen bubbles (Figure 3.2). This process can be shown by the following chemical equation (Pytlik & Saxena 1992):

$$2Al + 3Ca\left(OH\right)_2 + 6H_2O \rightarrow 3CaO \cdot Al_2O_3 \cdot 6H_2O + 3H_2$$

Aluminum Powder + Hydrated Lime → Tricalcium Hydrate + Hydrogen

The hydrogen that is formed in this process bubbles up out of the mixture and is replaced by air (www.gmchomesfl.com). The hydrogen, which is a lighter gas, rises and is replaced by air which is a denser gas that gets into the mix as the hydrogen foams up out of the material. The aluminum expansion agent is thoroughly mixed into the batch so that it is evenly distributed during the mixing process. The creation of hydrogen bubbles causes the mix to expand, increasing the volume of the mixture approximately two to five times its normal volume. The volume increase is dependent upon the amount of aluminum powder/paste that is introduced to react with the calcium hydroxide in the mixture. The less expansion that is induced will produce a higher strength material (more dense) versus the maximum amount of expansion induced, which produces a lower strength material (less dense). The microscopic voids created by the gas bubbles give AAC its light weight and other beneficial material properties, such as its high thermal resistance properties.

Figure 3.2: Air voids in AAC (Tanner 2003)

Step 3: Expansion, shaping, pre-curing, and cutting.

After the addition of the expansion agent, the mix is poured into metal molds where it is allowed to expand. If a plank or panel is being cast, then steel reinforcement is placed in the mold prior to pouring the mix into the mold. The steel reinforcement is used to give tension strength to the lightweight concrete material. When the mix is poured into the forms, commonly 20 feet x 4 feet x 2 feet thick (Pytlik & Saxena 1992), it first expands and then is allowed to pre-cure for several hours. The pre-curing stage is to allow enough time such that the block can maintain its shape outside of its mold. The pre-cured block can then be cut, utilizing a device that uses thin wires, into the desired shapes. Standard AAC masonry can be found with nominal dimensions of 8 inches deep by 24 inches long with varying thickness of 4 inches to 12 inches. The larger blocks are cut into solid masonry blocks similar to concrete masonry units (CMUs). Unlike CMU, AAC masonry units are cut from the larger block rather than being formed individually. The production of a plank, which can have reinforcement cast in, is not cut from a large block. The waste that is produced from cuttings or any leftover bits can be reused in the original mixture as aggregate after being finely ground.

Step 4: Final curing utilizing an autoclave

As defined by dictionary.com unabridged v1.1, an autoclave is “a strong, pressurized, steam-heated vessel.” This large steam-heated vessel is in effect a large pressure cooker by which the autoclaved aerated concrete is cured. Curing is the process by which the concrete mixture hardens through hydration (chemical process between cement and water), with the autoclave the blocks are cured with steam at high pressures. The pressure, temperature, and moisture are closely controlled for the twelve hours of curing time. The monitoring of proper pressure, temperature, and moisture allows for the optimum conditions for which hydration can occur. During this process the autoclave is heated to 374 degrees Fahrenheit and pressurized to 12 atmospheres of pressure, “quartz sand reacts with calcium hydroxide and evolves to calcium silica hydrate which account for the material's physical strength properties (www.gmchomesfl.com).” Basically, this step can be described as the actual baking portion like with bread.

Step 5: Packaging and shipping

After approximately twelve hours of curing time (Pytlik & Saxena 1992), the cured blocks are removed from the autoclave, packaged, and shipped. Figure 3.3 shows AAC being transported to a construction site. Various literature states that after AAC is autoclaved it can be immediately shipped and used for construction, it is assumed that the cooling step is not expressed as a period of time where the material is set aside for the express purpose to cool down, but as the period of time when the material is being packaged. At this point in the process the autoclaved aerated concrete units are ready for use in the construction process. Currently in the United States, the greatest production and use of AAC is in the southeast.

Figure 3.3: Transportation of the AAC to jobsite (www.gmchomesfl.com)

CHAPTER 4 - Material Properties

The material properties of autoclaved aerated concrete as listed by the 2005 MSJC Code are provided in the following sections:

4.1 AAC Material Properties

Autoclaved aerated concrete must have a minimum specified compressive strength (f'_{AAC}) of 290 psi. This is much lower than commonly specified f'_m of 1500 psi for CMU. The strength class of AAC materials is described in ASTM Specification C 1386.

Table 4.1: ASTM Specification C 1386 Autoclaved Aerated Concrete Masonry Units

Strength Class	Compressive Strength, psi (MPa)		Nominal Dry Bulk Density, lb/ft^2 (kg/m^3)	Density Limits, lb/ft^2 (kg/m^3)	Average Drying Shrinkage, %
	Average	Minimum			
PAAC-2	360 (2.5)	290 (2.0)	25 (400) 31 (500)	22 (350) – 28 (450) 28 (450) – 34 (550)	$\leq$ 0.02
PAAC-4	725 (5.0)	580 (4.0)	31 (500) 37 (600) 44 (700) 50 (800)	28 (450) – 34 (550) 34 (550) – 41 (650) 41 (650) – 47 (750) 47 (750) – 53 (850)	
PAAC-6	1090 (7.5)	870 (6.0)	37 (600) 44 (700) 50 (800)	35 (550) – 41 (650) 41 (650) – 47 (750) 47 (750) – 53 (850)	

Although the compressive strength of this material is much lower than standard CMU, the strength is adequate for a low-rise construction. The higher a building is constructed the more load the bottom portions of the structure must support. Because of the lower strength of masonry compared to steel or concrete, a masonry structure would need larger members sizes at the bottom to support the same loads and remain stable. This is why load-bearing masonry structures, and especially AAC, are not very tall when compared to buildings of steel and or concrete. The compressive strength of AAC is also adequate for the other uses such as partitions or curtain walls, as shown in Figure 4.1.

Figure 4.1: AAC used as an exterior non-bearing wall in Warsaw, Poland (Stephens)

The compressive strength of the grout (f'_g) used with construction of autoclaved aerated concrete must be within a range of 2000 psi to 5000 psi. This is the same grout that is used with CMU. See ASTM C 476 the Specification for Grout for Masonry. The same mortar types used for CMU are also used for AAC, however due to dimensional tolerances the AAC can be laid with thinner joints. ASTM C 270 is the specification for mortar for unit masonry.

4.1.1 Material Properties of AAC Masonry from the 2005 MSJC Code

The proceeding information are the material properties that apply to equations used to calculate the AAC cracking moment, shear strength, and the reinforcement yield strength used in AAC masonry construction.

The equation for masonry splitting tensile strength (2005 MSJC Code eq. A-1) which is used to determine the modulus of rupture is:

$$f_{tAAC} = 2.4 \times \sqrt{f'_{AAC}} \qquad \text{(Equation 4.1)}$$

For example, using the minimum compressive strength, f_{tAAC}, of 290 psi is:

$$f_{tAAC} \leq 2.4 \times \sqrt{290\,psi} = 40.87\,psi$$

The modulus of rupture for autoclaved aerated concrete is twice the masonry splitting tensile strength with maximum limitations of 50 psi and 80 psi for sections that contain horizontal leveling bed and thin-bed mortar, respectively. A horizontal leveling bed is a thicker layer of mortar that is used to even out or level the height of a masonry unit, whereas thin-bed mortar are thinner layers of mortar that can be used with AAC. The modulus of rupture can be calculated as:

$$f_{r\,AAC} = 2 \times f_{tAAC} \qquad \text{(Equation 4.2)}$$

Masonry direct shear strength is calculated using 2005 MSJC Code eq. A-2.

$$f_v = 0.15 \times f'_{AAC} \qquad \text{(Equation 4.3)}$$

For example, using the minimum compressive strength, f_v can be calculated as:

$$f_v \le 0.15 \times 290\,psi = 43.5\,psi$$

Depending on whether or not mortar is used in the bed joint, the coefficient of friction between AAC units is given as the following and can be found in MSJC Code 2005 A.1.8.5:

$\mu = 0.75$ ~ friction between AAC units

$\mu = 1.00$ ~ friction when using thin bed or leveling bed mortar

The coefficient of friction is used in the calculation of the sliding shear capacity which will be covered in chapter 5.

The maximum yield strength, f_y, of the reinforcing steel used in AAC is 60,000 psi. This refers to the steel used to resist shear and tensile stresses that exceeds the strength of AAC.

The modulus of elasticity of AAC can be found in the 2005 MSJC Code, section 1.8.2.3.1 and is taken as:

$$E_{AAC} = 6500\left(f'_{AAC}\right)^{0.6} \qquad \text{(Equation 4.4)}$$

This equation for modulus of elasticity is found in the 2005 MSJC Code, section 1.8.2.3.1.

The bearing strength of AAC is a strength reduction factor, 0.6, multiplied by the compressive strength (f'_{AAC}) multiplied by the area as defined in the 2005 MSJC Code Section A.1.10.2. The nominal bearing strength (C_n) can be described in the following equation:

$$C_n = Af'_{AAC} \qquad \text{(Equation 4.5)}$$

The area used for the nominal bearing strength is:

$$A_1 = \textit{direct bearing area}$$

or:

$$A_1\sqrt{\frac{A_2}{A_1}} \leq 2A_1$$

"Where A2 is the lower base of the largest frustum of a right pyramid or cone having A1 as the upper base sloping at 45 degrees from the horizontal and wholly contained by the support." (2005 MSJC Code) This terminates at head joint for walls not in running bond.

4.2 AAC Performance Properties

There are other properties of autoclaved aerated concrete that can be described as material performance properties as contrasted with strength of material properties. Material performance properties describes the less structural properties of AAC.

Normally, concrete weighs in at about 130 lbs to 155 lbs per cubic foot whereas AAC weighs in, at its lowest, at around 25 lbs per cubic foot and at its highest at around 50 lbs per cubic foot. This makes for easy transport (more material can be transported at once) and installation (faster installation of lighter masonry units). This idea is further discussed in chapter 7.

AAC has good thermal properties without the aid of insulation. AAC can absorb large amounts of radiant energy and slowly releases that thermal energy to the surroundings. An 8 inch wall constructed of AAC has an R-Value approximately of 11-12 (Aercon) without considering any other materials that may be attached for finishes. In comparison, 1 inch rigid insulation (cellular polyurethane) has an approximate R-Value of 6. The higher the R-Value is, the better the thermal properties. An 8 inch CMU Wall with cells filled with vermiculite has an approximate R-Value of 1.5. These R-Values are taken from the *Principles of Heating Ventilation and Air Conditioning* (Principles of HVAC 2001). A 2 inch x 6 inch wood stud wall with R-13 batt insulation, wood sheathing, gypsum board, felt, and shingles has an R-value of approximately 15.23 (Albright, Gay, Stiles, Worman & Zak, 1980). This is an example of a wall combining structural components with non structural materials to form something with an insulating value that meets requirements. In contrast AAC alone acts as the structural component and insulating material.

In addition to its thermal insulation properties, is AAC's 4 hour fire rating (Pytlik & Saxena, 1992). Both 6 inch load bearing walls and 4 inch non load bearing walls of AAC have 4 hour fire ratings. This also includes 6 inch roof and floor panels. (Aercon) In comparison, a wall of CMU has a required thickness of 8 inches or more to obtain a 4 hour fire rating (NCMA TEK 7-1A). Concrete is a noncombustible material that is commonly used for fire separation walls as solid normal weight and light weight concrete, CMU and AAC.

AAC is good as a sound absorber and has been used frequently as sound walls along side roadways. A material that is a good sound absorber has the capacity to reduce reflected sound by absorbing some of the sound without all of it being reflected back. On the other hand, AAC transmits sound at a somewhat higher rate than normal CMU. The sound transmission class, STC, of an 8 inch AAC wall is 41(www.acsolar.com), which is comparable to an STC value of 49 for a hollow 8 inch CMU wall (NCMA TEK 13-1a). The higher the STC is, the greater the sound reduction or the lower the STC the less sound reduction.

Because AAC is a non-organic building material, autoclaved aerated concrete is naturally mold resistant. It is also unaffected by termites and does not decompose. This makes AAC more of a low maintenance material than wood.

Through its various material and performance properties it is seen that AAC is a great material for the construction of walls in buildings even though it has a relatively low compressive strength. It is easily installed because of its light weight, has good thermal characteristics which increase a building's energy efficiency, a 4-hour fire rating, and is a non-organic material produced from readily available material. Its strength, although lower than standard CMU, is quite adequate for low-rise construction.

CHAPTER 5 - Structural Design Requirements

In the design of AAC structures, the requirements set forth in the 2005 Masonry Standards Joint Committee Code, Specification, and Commentaries Appendix A is used. This chapter provides several design examples demonstrating various code requirements for the design of AAC as a structural building material. Members under axial compression, bending, combined axial compression and bending, out of plane and in plane loading of walls by wind or seismic forces, are covered. The design of AAC currently only uses Strength Design.

5.1 Unreinforced and Reinforced Members Subjected to Axial Compression Only

AAC members that are subjected to axial compression consist mainly of bearing walls. For the design of unreinforced AAC members the nominal axial strength of a member is based on the compressive strength of the material and the slenderness of the wall. The slenderness of the wall is determined by taking the height of the member divided by the member's radius of gyration. The radius of gyration of a member is the square root of the moment of inertia divided by the area of its cross-section or:

$$r = \sqrt{\frac{I}{A}} \qquad \text{(Equation 5.1.1)}$$

Once the slenderness ratio, h/r is determined, the code specifies two formulas to determine the axial strength of a member depending on the slenderness ratio. If the ratio is equal to or less than 99 the nominal axial compressive strength is given by 2005 MSJC Code (Eq. A-3):

$$P_n = 0.80 \times \left[0.85 \times A_n \times f'_{AAC} \times \left(1 - \left(\frac{h}{140r} \right)^2 \right) \right] \qquad \text{(Equation 5.1.2)}$$

For a slenderness ratio greater than 99 the nominal axial compressive strength is given by 2005 MSJC Code (Eq. A-4):

$$P_n = 0.80 \times \left[0.85 \times A_n \times f'_{AAC} \times \left(\frac{70r}{h} \right)^2 \right] \qquad \text{(Equation 5.1.3)}$$

In the following example with the conditions shown, the dimensions of the unreinforced AAC wall are used to determine the maximum factored load per linear foot of wall (w_u) the wall can support:

- Height of wall is 10’
- Wall thickness 8”
- $f'_{AAC} = 290\,psi$
- Properties are based on solid block

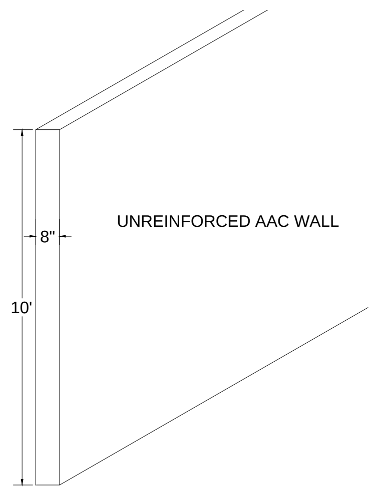


Figure 5.1: Example Unreinfored AAC Wall Axial Strength

Moment of Inertia per foot length of wall:

$$I = \frac{bh^3}{12} = \frac{12\times\left(8\right)^3}{12} = 512in^4$$

Area per foot length of wall:

$$A = bh = 8\times 12 = 96in^2$$

Radius of gyration:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{512in^4}{96in^2}} = 2.309in$$

Slenderness ratio:

$$\frac{h}{r} = \frac{\left(10\times 12\right)in}{2.309in} = 51.97$$

$\frac{h}{r} = 51.97 \le 99$ Therefore use Equation 5.1.2

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

$$P_n = 0.80 \times \left[0.85 \times 96in^2 \times 290\,psi \times \left(1 - \left(\frac{(10 \times 12)in}{140 \times 2.309in} \right)^2 \right) \right] = 16322.4lbs$$

The nominal axial compressive strength that was found in this example is based upon a one foot section of wall. Therefore the ultimate load the wall can support is the nominal axial strength per foot of wall multiplied by the strength reduction factor, 0.9. The strength reduction factor can be found in the 2005 MSJC Code Section A1.5.1.

$$P_n = 16322.4\,{}^{lbs}\!/_{ft}$$

$$\phi P_n = 0.9(16322.4lbs) = 14690\,{}^{lbs}\!/_{ft}$$

$$w_u = 14690\,plf$$

This is the calculation for unreinforced AAC under axial loading. The calculation for reinforced AAC subjected to axial loading is slightly different using the 2005 MSJC Code. In reinforced AAC, reinforcing steel or welded wire fabric can be placed in the form before the molding process for horizontal or vertically spanning wall panels. In the case of AAC masonry block, hollow cells can be cut in the block so that rebar can be placed in a cell and be grouted during the construction process. Reinforcement can provide additional strength in compression for AAC but does not usually apply because the reinforcement must be confined by ties. This is similar to reinforced CMU. As in the previous calculations, the slenderness ratio is used to determine which equation is to be used in calculating the nominal axial strength of the AAC in compression. If the slenderness ratio is equal to or less than 99 the nominal axial strength is given by 2005 MSJC Code (Eq. A-7)

$$P_n = 0.80\left[0.85 \times f'_{AAC} \times (A_n - A_s) + f_y \times A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \text{(Equation 5.1.4)}$$

For a slenderness ratio greater than 99 the nominal axial strength is given by 2005 MSJC Code (Eq A-8)

$$P_n = 0.80\left[0.85 \times f'_{AAC} \times (A_n - A_s) + f_y \times A_s\right]\left(\frac{70r}{h}\right)^2 \quad \text{(Equation 5.1.5)}$$

In reinforced AAC, the area of steel has an effect on the overall nominal capacity of the member. The reinforcing displaces a small area of concrete in the AAC member and so that area is taken out for the calculation of the nominal strength. In the following example the nominal axial strength of a reinforced wall is calculated using the given conditions:

- Wall height is 12'
- Nominal wall thickness is 6"
- Wall has vertical #5 bars placed at 60" on center
- $f'_{AAC} = 290\,psi$
- $f_y = 60000\,psi$

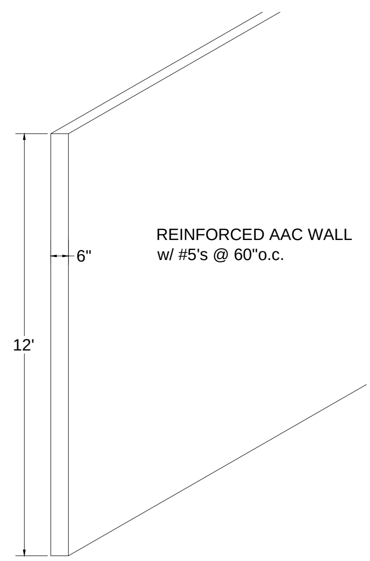


Figure 5.2: Example Problem Reinforced AAC Wall Axial Strength

Moment of Inertia per foot length of wall:

$$I = \frac{bh^3}{12} = \frac{12\times(6)^3}{12} = 216in^4$$

Area per foot length of wall:

$$A = bh = 6\times 12 = 72in^2$$

Radius of gyration:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{216in^4}{72in^2}} = 1.732in$$

Slenderness ratio:

$$\frac{h}{r} = \frac{(12 \times 12)in}{1.732in} = 83.1$$

$\frac{h}{r} = 83.1 \leq 99$ Therefore use Equation 5.4

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

$$f'_{AAC} = 290psi$$

$$A_n = 72\, in^2/ft \left(\frac{60in}{12\, in/ft}\right) = 360in^2$$

$$A_s = 0.31in^2$$

$$f_y = 60ksi = 60000psi$$

$$P_n = 0.80\left[0.85 \times 290psi \times (360in^2 - 0.31in^2) + 60000psi \times 0.31in^2\right]\left[1 - \left(\frac{(12 \times 12)in}{140(1.732in)}\right)^2\right]$$

$$P_n = 55,550lbs$$

This gives the nominal axial strength per 5 feet of wall. From this the ultimate load is determined as the nominal axial strength times the strength reduction factor divided by 5'.

$$P_n = \frac{55,550lbs}{5\,ft} = 11,110plf$$

$$\phi P_n = 0.9(11110lbs) = 9999plf$$

$$w_u = 9999plf$$

The reinforced nominal strength can be rewritten, to determine how much steel will be required and at what spacing to help support the axial load on the member. It can also be rewritten so that the required thickness of the wall can be determined based on a given area of steel. The design strength is determined by multiplying the nominal strength, P_n, by the strength reduction factor, ϕ, found in section A.1.5 of the 2005 MSJC Code. From A.1.5, the strength reduction factor for axially loaded reinforced AAC is 0.90 and the factor for unreinforced AAC is 0.60. The factored axial load must always be less than or equal to the design strength:

$$\phi P_n \geq P_u \qquad \text{(Equation 5.1.6)}$$

These preceding provisions are for axial loads only and the vertical reinforcing must be tied similar to a column. Effective ties for compression steel in walls are not normal practice for masonry walls; therefore, most walls are designed as unreinforced even though vertical reinforcing is provided. In section 5.3 of this report, the design strength for combined axial and bending will presented.

5.2 Members Subjected to Flexure

In most cases, a member designed for flexure must be reinforced to resist tension forces because AAC has a very low strength in tension. The material covered in this section is only applicable to members that are subjected to flexure combined with an axial compressive force of less than five percent of the net cross-sectional area of the member multiplied by f'_{AAC} as given in the 2005 MSJC Code, Section A.3.4.2.1. After it is determined that the axial force in the member is less than the maximum allowed, the nominal moment strength is determined. The nominal flexural strength of a beam must be greater than or equal to 1.3 multiplied by the nominal cracking moment strength, M_{cr}.

$$M_n \geq 1.3 M_{cr} \qquad \text{(Equation 5.2.0)}$$

The 2005 MSJC Code does not directly address the design of flexural steel in an AAC member for purely flexural members. The formulas to determine the steel in beams was derived from the nominal moment capacity equation for walls subjected to out of plane bending and axial loads. For the case of purely axial loads the portions for moment produced by axial load at an eccentricity and axial load with a deflection effect are taken as zero. The 2005 MSJC (Eq. A-20 and A-21) are used to determine nominal flexural strength are:

$$M_n = \left(A_s f_y + P_u\right) \times \left(d - \frac{a}{2}\right) \qquad \text{(Equation 5.2.1)}$$

$$a = \frac{\left(A_s f_y + P_u\right)}{0.85 f'_{AAC}\, b} \qquad \text{(Equation 5.2.2)}$$

Modified for flexure only, the equations become:

$$M_n = \left(A_s f_y\right) \times \left(d - \frac{a}{2}\right) \qquad \text{(Equation 5.2.3)}$$

Where

$$a = \frac{\left(A_s f_y\right)}{0.85 f'_{AAC}\, b} \qquad \text{(Equation 5.2.4)}$$

In the following flexure example with the given conditions, the minimum and maximum steel is determined followed by the calculation of the design bending strength for the beam:

- Beam span is 6'
- Nominal width, b, is 8"
- Nominal depth, h, is 16"
- Nominal depth of steel, d, is 12"
- $f'_{AAC} = 290\,psi$
- $f_y = 60ksi$

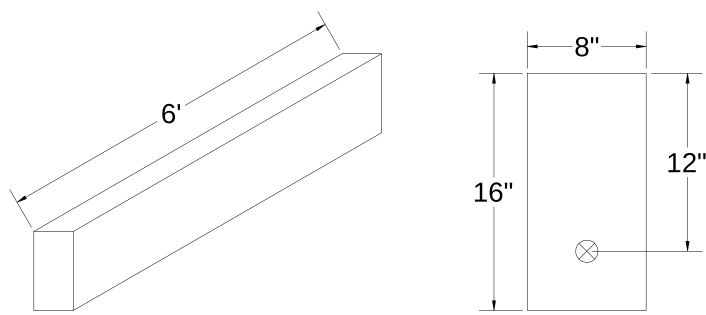


Figure 5.3: Beam Example Flexural Strength

$$M_n \geq 1.3 M_{cr}$$

$$A_s f_y\left(d - \frac{a}{2}\right) \geq 1.3\frac{f_{rAAC} I_g}{\bar{y}}$$

$$A_{s\min} f_y\left(d - \frac{a}{2}\right) = 1.3\frac{f_{rAAC} I_g}{\bar{y}}$$

$$A_{s\min} = 1.3\frac{f_{rAAC} I_g}{\bar{y} f_y\left(d - \frac{a}{2}\right)} \qquad \text{(Equation 5.2.5)}$$

$$f_{rAAC} = 2\left(2.4\sqrt{f'_{AAC}}\right) = 2\left(2.4\sqrt{290\,psi}\right) = 81.74\,psi \qquad \text{(Equation 4.2)}$$

$$\bar{y} = \frac{h}{2} = \frac{16in}{2} = 8in$$

$$I_g = \frac{bh^3}{12} = \frac{8\times16^3}{12} = 2730.67in^4$$

$$a = \frac{0.67d\left(0.003\right)}{0.003+0.00207} = \frac{0.67\left(12in\right)\left(0.003\right)}{0.003+0.00207} = 4.757in$$

$$A_{s\min} = 1.3\frac{81.74\,psi\times2730.67in^4}{8in\times60000\,psi\left(12in-\dfrac{4.757in}{2}\right)} = 0.063in^2$$

Maximum Steel is based on the 2005 MSJC Commentary 3.3.3.5 the value 0.85, 0.67, and the strain value comes from 2005 MSJC Code Section A.3.3.5.

$$A_{s\max} = \frac{0.85\times0.67\,f'_{AAC}\dfrac{\varepsilon_{mu}}{\varepsilon_{mu}+\alpha\varepsilon_y}bd}{f_y} \qquad \text{(Equation 5.2.6)}$$

$$\alpha = 1.5 \ \left(tension\ reinforcement\ \ factor\right)$$

$$\varepsilon_{mu} = 0.003 \ \left(\max\ \ usable\ stain\ in\ masonry\right)$$

$$\varepsilon_y = 0.00207 \ \left(yield\ \ strain\right)$$

$$A_{s\max} = \frac{0.85\times0.67\left(290\,psi\right)\dfrac{0.003}{0.003+\left(1.5\times0.00207\right)}8in\times12in}{60000\,psi} = 0.16in^2$$

Note: The tension reinforcement factor, α, can be found in MSJC Commentary 3.3.3.5. Therefore a bar with area of steel between the minimum and maximum will be selected. A #3 bar with an area of 0.11 in^2 is the only one which meets the requirements.

$$A_s = 0.11in^2$$

Using the modified equations for flexure only:

$$a = \frac{\left(0.11in^2\times60000\,psi\right)}{0.85\times290\,psi\times8in} = 3.347in$$

$$M_n = \left(0.11in^2\times60000\,psi\right)\times\left(12in-\frac{3.347in}{2}\right) = 68154.9lb\cdot in = 5679.6lb\cdot ft$$

5.3 Members Subjected to Combined Axial and Bending

In this section the concepts of the two previous sections will be combined. This section is based on walls that have out of plane loading plus an axial compression force. The nominal axial strength is the same as discussed in the previous section for members with axial loading only. For the nominal flexural capacity the provisions in 2005 MSJC Code A.3.5 are used. Equation 5.1.4 (A-7 of the 2005 MSJC Code) is used for walls with a slenderness ratio less than or equal to 99. Equation 5.1.5 (A-8 of the 2005 MSJC Code) is used for walls with a slenderness ratio greater than 99. These equations give the nominal axial compressive strength. The nominal flexural strength is determined using equation 5.2.1 and equation 5.2.2. The ultimate axial load (P_u) is taken as the summation of the factored wall weight at mid-height and the factored load on the wall, as given in equation 5.3.1 (2005 MSJC Code Eq. A-18).

$$P_u = P_{uw} + P_{uf} \qquad \text{(Equation 5.3.1)}$$

The nominal axial strength must further meet the requirements of equation 5.3.2(2005 MSJC Code Eq. A-16).

$$\frac{P_u}{A_g} \leq 0.2 f'_{AAC} \qquad \text{(Equation 5.3.2)}$$

If the wall being designed does not meet the requirement of equation 5.3.2 then the provisions of the 2005 MSJC Code A.3.5.5 must be used. Section A.3.5.5 also applies if the slenderness ratio is greater than 30, if the slenderness of 30 is exceeded a minimum 6 inch wall thickness is required. For the following given data on the wall, the nominal axial strength and nominal flexural strength can be determined. Afterward using the nominal flexural strength the maximum out of plane loading can be determined (For example this could be out of plane wind load):

- Wall height is 12’
- Nominal wall thickness is 8”
- A #4 bar is placed vertically every 24” (2’-0”) in the center of the wall.
- The density of the wall is 35 pounds per cubic foot
- A superimposed factored load of 1000 pounds per linear foot at center line of wall. This load is assumed to be from a combination of dead load and snow load.
- Out of plane wind factored load of 10 pounds per square foot
- $f'_{AAC} = 290\,psi$
- $f_y = 60ksi$

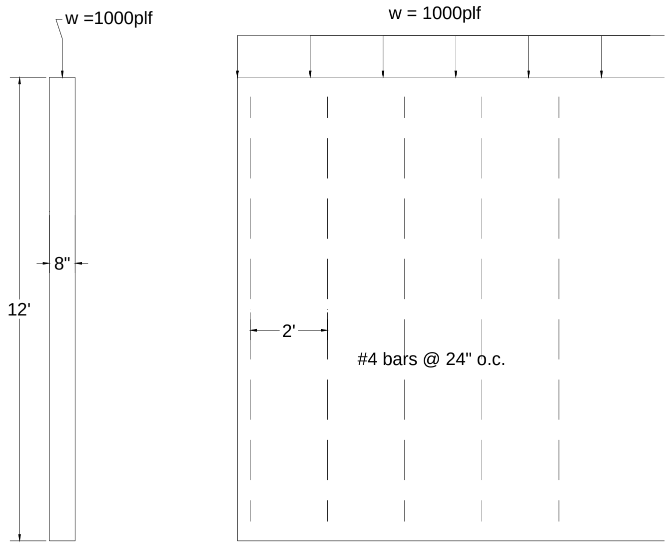


Figure 5.4: AAC Wall Example Axial and Bending Strength

The factored axial load on the wall at mid-height is determined since it will be combined with the maximum moment which will occur at mid-height.:

$$P_u = P_{uw} + P_{uf}$$

$$P_{uw} = (1.2)\frac{1}{2}\times 35\,pcf\left(\frac{8"}{12}\times 12'\right) = 168\,plf$$

This uses a 1.2 load factor on the dead load assuming a load combination$1.2D + 1.6S$ +0.8W.

$$P_u = 168\,plf + 1000\,plf = 1168\,plf$$

Check requirements of equation 5.3.2.

$$\frac{P_u}{A_g} = \frac{1168\,plf}{(8in\times 12in)\,{in^2}\!/\!{ft}} = 12.17\,psi \le 0.2\,f'_{AAC} = 0.2(290\,psi) = 58\,psi \qquad \text{OK}$$

Moment of Inertia per foot length of wall:

$$I = \frac{bh^3}{12} = \frac{12\times(8)^3}{12} = 512in^4$$

Area per foot length of wall:

$$A = bh = 8\times 12 = 96in^2$$

Radius of gyration:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{512in^4}{96in^2}} = 2.309in$$

Slenderness ratio:

$$\frac{h}{r} = \frac{12\,ft\left(12\,{}^{in}\!/\!_{ft}\right)}{2.309in} = 62.36 \geq 30$$ A 6 inch minimum wall thickness is required.

$\frac{h}{r} \leq 99$ Therefore use equation 5.1.4.

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

$$f'_{AAC} = 290\,psi$$

$$A_n = 96in^2\left(\frac{24in}{12}\right) = 192in^2$$

$$A_s = 0.20in^2$$

$$f_y = 60000\,psi$$

Since the steel is not confined and therefore must be neglected for the calculation of compressive strength (this would be the same as using Equation 5.1.2 versus 5.1.4)

$$P_n = 0.80\left[0.85 \times 290\,psi \times \left(192in^2\right)\right]\left[1 - \left(\frac{12 \times 12}{140\left(2.309\right)}\right)^2\right]$$

$$P_n = 30349\,{}^{lbs}\!/\!_{2\,ft} \Rightarrow P_n = 15174.5\,plf$$

$$\phi = 0.9$$

$$\phi P_n = 0.9\left(15174.5\,plf\right) = 13657\,plf$$

$$\phi P_n = 13657\,plf \geq P_u = 1168\,plf$$ OK

Determining the nominal flexural strength Equation 5.2.1 and 5.2.2:

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

$$a = \frac{\left(A_s f_y + P_u\right)}{0.85 f'_{AAC}\, b} = \frac{\left(0.20in^2\left(60000\,psi\right) + 1168\,plf\left(2\,ft\right)\right)}{0.85\left(290\,psi\right)\left(8in\right)} = 7.27in$$

$$d = 4in$$

$$M_n = \left(0.2in^2\left(60000psi\right) + 1168plf\left(2ft\right)\right) \times \left(4in - \frac{7.27in}{2}\right) = 5232.64lb \cdot in$$

$$M_n = 436lb \cdot ft$$

$$\phi = 0.9$$

$$\phi M_n = 0.9\left(436lb \cdot ft\right) = 392.4lb \cdot ft$$

Comparing the moment from out of plane wind:

$$M_u = \frac{wL^2}{8} = \frac{10psf\left(1ft\right)\left(12ft\right)^2}{8} = 180lb \cdot ft \le \phi M_n = 392.4lb \cdot ft \qquad O.K.$$

Equations 5.3.3 (2005 MSJC Code equation A-17) for the total moment on the wall takes into account the effects of axial load eccentricity and second order P-delta effects.

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad \text{(Equation 5.3.3)}$$

$w_u =$ *factored uniform load on wall*

$P_{uf} =$ *factored load from roof (floor)*

$e_u =$ *eccentricity of* P_{uf}

$\delta_u =$ *deflection from factored moment*

The deflection in the preceding equation can be found using the deflection calculations presented in section 5.5 of this chapter, the only difference being that a factored moment, M_u, is used instead of the service moment, M_{ser}. For this example this effect was not taken into account because the nominal flexural strength was the desired result, when actual loads are being used equation 5.3.3 will be used.

The area of steel can be checked against the maximum area prescribed in the 2005 MSJC Code A.3.3.5 as:

$$A_{s\max} = \frac{0.85f'_{AAC}\left(0.67d\right)\left(\frac{0.003}{1.5\left(0.00207\right)+0.003}\right)b - \frac{P_u}{\phi}}{f_y} \qquad \text{(Equation 5.3.4)}$$

5.4 Members Subjected to Shear

This section describes the process needed to determine the nominal shear strength, V_n, of a member. The nominal shear strength can be described as the combination of the nominal shear strength of the AAC itself, V_{AAC}, plus the nominal shear strength provided by the steel reinforcement, V_s (2005 MSJC Code equation A-9).

$$V_n = V_{AAC} + V_s \qquad \text{(Equation 5.4.1)}$$

The nominal shear strength is limited by two equations (2005 MSJC Code equation A-10 and A-11), or an interpolation between them, based on the ratio of the ultimate moment and the ultimate shear multiplied by the depth of the member in the direction which shear is considered.

For:

$$\frac{M_u}{V_u d_v} \le 0.25 \qquad \text{(Equation 5.4.2)}$$
$$\Rightarrow V_n \le 6A_n\sqrt{f'_{AAC}}$$

And for:

$$\frac{M_u}{V_u d_v} \ge 1.00 \qquad \text{(Equation 5.4.3)}$$
$$\Rightarrow V_n \le 4A_n\sqrt{f'_{AAC}}$$

V_{AAC} is determined based on the provisions in sections A.3.4.1.2.1 to A.3.4.1.2.5 of the 2005 MSJC Code. The minimum value for shear strength of AAC is used as the most critical.

In-Plane Shear Strength:

For nominal shear capacity governed by web shear cracking, the following equations are to be used. Eq. 5.4.4a is used for AAC masonry with mortared head joints, Eq. 5.4.4b is used for AAC masonry without mortared head joints, and Eq. 5.4.4c is to be used for AAC masonry in other than running bond (2005 MSJC Code equation A-12(a, b, c)).

$$V_{AAC} = 0.95 l_w t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,l_w t}} \qquad \text{(Equation 5.4.4a)}$$

$$V_{AAC} = 0.66 l_w t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,l_w t}} \qquad \text{(Equation 5.4.4b)}$$

$$V_{AAC} = 0.9\sqrt{f'_{AAC}}\,A_n + 0.05P_u \qquad \text{(Equation 5.4.4c)}$$

$l_w = length\ of\ wall\ considered\ in\ direction\ of\ shear$

$t = thickness\ of\ wall$

For nominal shear strength governed by crushing of the diagonal compressive strut, the following equation is used (2005 MSJC Code equation A-13a).

When:

$$\frac{M_u}{V_u d_v} < 1.5$$

$$\Rightarrow V_{AAC} = 0.17 f'_{AAC}\, t \frac{h l_w^2}{h^2 + \left(\frac{3}{4} l_w\right)^2} \qquad \text{(Equation 5.4.5)}$$

When:

$$\frac{M_u}{V_u d_v} > 1.5$$

V_{AAC} governed by crushing of the diagonal strut need not be considered. This is not taken as shear strength of zero. Instead it is not included in determining the critical shear strength.

For a nominal shear strength governed by sliding shear at an un-bonded surface (2005 MSJC Code equation A-13b):

$$V_{AAC} = \mu_{AAC} P_u \qquad \text{(Equation 5.4.6)}$$

The coefficient of friction given previously in Chapter 4 is:

$\mu_{AAC} = 0.75$ ~ For AAC to AAC

$\mu_{AAC} = 1.00$ ~ When thin bed mortar or leveling bed mortar is used

Out-of-Plane Shear Strength:

For nominal shear strength by out-of-plane loading on a wall (2005 MSJC Code equation A-15).

$$V_{AAC} = 0.8\sqrt{f'_{AAC}}\, bd \qquad \text{(Equation 5.4.7)}$$

Shear Strength from Steel:

The second part of the nominal shear strength equation is the strength provided by reinforcing steel in the section considered (2005 MSJC Code equation A-14).

$$V_s = \frac{A_v}{s} f_y d_v \qquad \text{(Equation 5.4.8)}$$

Where:

$$A_v = Area\ of\ shear\ steel$$
$$f_y = Yield\ stress\ of\ steel$$
$$s = spacing\ of\ shear\ steel$$
$$d_v = depth\ in\ the\ direction\ shear\ considered$$

An example demonstrating how to determine in-plane and out-of-plane shear strength of a non-bearing wall is provided for the following conditions:

- Wall height is 10’
- Wall is composed of 8” AAC Masonry in running bond with mortared head joints
- Thin-bed mortar is used
- #5 bars are placed horizontally in fully grouted bond beams every 32”
- Vertical #4 bars are spaced evenly every 24”
- Wall length is 20’
- The 8”x8”x16” bond beam blocks have a 1” face shell
- Grout strength is 2000 psi
- Unit weight of grouted block is 120 pcf, other block is 35 pcf
- A conservative value of $\frac{M_u}{V_u d_v} = 1.0$ is assumed
- $f'_{AAC} = 290\,psi$
- $f_y = 60ksi$

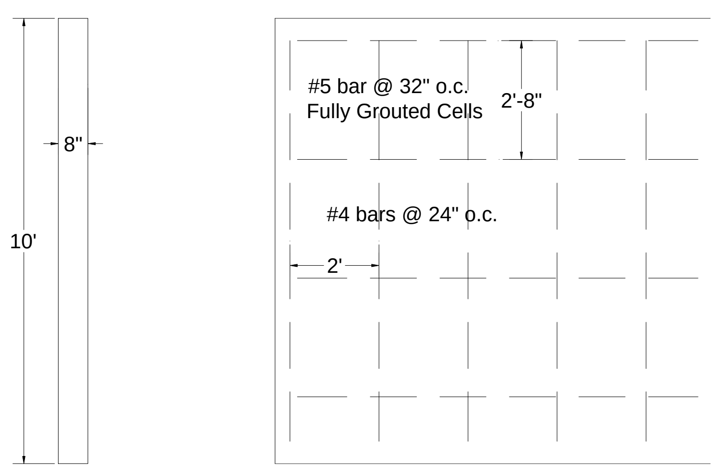


Figure 5.5: Wall Example In Plane and Out of Plane Shear

Determine the nominal shear strength based on the limit states listed above.

In-Plane Shear Strength:

Find V_{AAC} for web shear cracking:

The load provided by the upper half of the wall can be determined as:

$$P_u = 5.5rows\left[35\,pcf\left(\frac{8in\times 8in}{144\,{}^{in^2}\!/\!_{ft^2}}\right)\right] + 2rows\left[120\,pcf\left(\frac{8in\times 8in}{144\,{}^{in^2}\!/\!_{ft^2}}\right)\right] = 192.22\,plf$$

Assuming a load case of $1.2D + 1.6W + 0.5S$, the ultimate axial load becomes:

$$P_u = 1.2(192.22\,plf) = 231\,plf$$

For mortared head joints use equation 5.4.4a:

$$V_{AAC} = 0.95 l_w t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{2.4\sqrt{f'_{AAC}}\,l_w t}}$$

$$l_w = 20\,ft\left(12\,{}^{in}\!/\!_{ft}\right) = 240in$$

$$V_{AAC} = 0.95(240in)(8in)\sqrt{290\,psi}\sqrt{1+\frac{231\,plf\,(20\,ft)}{2.4\sqrt{290\,psi}\,(240in)(8in)}} = 31962.9lbs$$

The value $\frac{M_u}{V_u d_v} = 1.0$ is assumed, therefore use equation 5.4.5 for crushing of the diagonal strut:

$$V_{AAC} = 0.17 f'_{AAC}\, t\frac{h l_w^2}{h^2 + \left(\frac{3}{4}l_w\right)^2}$$

$$V_{AAC} = 0.17(290\,psi)(8in)\frac{\left(10\,ft\times 12\,{}^{in}\!/\!_{ft}\right)(240in)^2}{\left(10\,ft\times 12\,{}^{in}\!/\!_{ft}\right)^2 + \left(\frac{3}{4}(240in)\right)^2} = 47328lbs$$

For sliding shear using equation 5.4.6:

$$V_{AAC} = \mu_{AAC} P_u$$

$$\mu_{AAC} = 1.0$$

$$V_{AAC} = 1.0\left(231\,plf\,(20\,ft)\right) = 4620lbs$$

In-plane nominal shear strength is governed by the smaller value from sliding shear, this would not be the case if a large enough axial load was applied to the wall. If the load that a wall is subjected to becomes high then the crushing of the diagonal strut would be the controlling case. The shear strength considering the horizontal reinforcing steel using equation 5.4.8:

$$V_s = \frac{A_v}{s} f_y d_v$$
$$A_v = 0.31 in^2$$
$$s = 32 in$$
$$d_v = 236 in$$
$$V_s = \frac{0.31 in^2}{32 in}(60000\,psi)(236 in) = 137175 lbs$$

The nominal shear strength using equation 5.4.1 for in plane forces is:

$$V_n = V_{AAC} + V_s$$
$$V_n = 4620 lbs + 137175 lbs = 141795 lbs$$
$$\phi = 0.8$$
$$\phi V_n = 0.8(141795 lbs) = 113436 lbs$$

Using equation 5.4.1 for out of plane shear strength:

$$V_{AAC} = 0.8 bd\sqrt{f'_{AAC}}$$
$$V_{AAC} = 0.8(24 in)(4 in)\sqrt{290\,psi} = 1308 lbs$$
$$V_n = V_{AAC}$$
$$V_n = 1308 lbs$$
$$\phi = 0.80$$
$$\phi V_n = 0.80(1308 lbs) = 1046.4 lbs$$

Check versus equation 5.4.3:

$$\frac{M_u}{V_u d_v} = 1.0 \geq 1.00$$
$$\Rightarrow V_n \leq 4 A_n \sqrt{f'_{AAC}}$$
$$V_n = 4(24 in \times 8 in)\sqrt{290\,psi} = 6539 lbs$$
$$\phi V_n = 0.8(6539 lbs) = 5231.2 lbs$$
$$\phi V_n = 1046.4 lbs \leq 5231.2 lbs \Rightarrow O.K.$$
$$\Rightarrow \phi V_n = 1046.4 lbs$$

Maximum out of plane design shear strength:

$$\phi V_n \geq V_u$$

$$sL = \textit{area under loading}$$

$$\Rightarrow \frac{2(1046.4lbs)}{\left(24in \times {}^{1\,ft}\!/_{12in}\right)(20\,ft)} = 52.32\,psf$$

This gives the maximum factored out of plane load in pounds per square foot.

Due to the fact that a grouted block may be used and grout has a higher strength than the AAC the nominal shear strength of the AAC will be based on the strength of the grouted block and therefore will use equation 5.4.9 to determine (2005 MSJC Code Equation 3-21) to determine the nominal shear strength of the grouted block (V_m).

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25P_u \qquad \text{(Equation 5.4.9)}$$

$$f'_m \Rightarrow f'_g$$

Also the strength of the steel is based on the normal masonry equation, equation 5.4.10 (2005 MSJC Code Equation 3-22).

$$V_s = 0.5\frac{A_v}{s} f_y d_v \qquad \text{(Equation 5.4.10)}$$

5.5 Deflection Limitations

The 2005 MSJC addresses the requirements for deflection limitations of vertical members, most typically walls and columns. The maximum allowable horizontal deflection of a vertical element is 0.7% of the member height, as described in equation 5.5.1 (2005 MSJC Code Equation A-22).

$$\delta_s \leq 0.007h \qquad \text{(Equation 5.5.1)}$$

For example, the horizontal deflection at mid-span for a 10’ tall wall cannot exceed 0.84 inches. The calculation for the deflection using section A.3.5.6 (2005 MSJC Code) is dependent upon whether of not the applied service load moment, M_{ser}, is less than the cracking moment. The cracking moment is calculated using equation 5.5.2 (2005 MSJC Code Eq. A-25).

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{(Equation 5.5.2)}$$

For the case where service load moment, M_{ser}, is less than the cracking moment, deflection is calculated using equation 5.5.3 (2005 MSJC Code Eq. A-23).

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

$$\Rightarrow \delta_s = \frac{5M_{ser}h^2}{48E_{AAC}I_g} \qquad \text{(Equation 5.5.3)}$$

When M_{ser} is less than the nominal moment but greater than the cracking moment, deflection is calculated using equation 5.5.4 (2005 MSJC Code Eq. A-24).

$$M_{cr} < M_{ser} < M_n$$

$$\Rightarrow \delta_s = \frac{5M_{ser}h^2}{48E_{AAC}I_g} + \frac{5\left(M_{ser} - M_{cr}\right)h^2}{48E_{AAC}I_g} \qquad \text{(Equation 5.5.4)}$$

The maximum deflection that a member can have is to be calculated using out of plane forces and axial forces that are placed at an eccentricity. Although examples are not provided, the provisions of this section will be used to check the deflection limitations of the example building design in Chapter 6.

CHAPTER 6 - Example Building Design

This building design example is provided to demonstrate the design requirements explained in Chapter 5. The building has 12 foot high AAC masonry walls with 10 inch AAC roof panels and plan dimensions of 30 feet wide by 50 feet in length. The roof panels used were determined using the Aercon Technical Manual (Aercon). The building is located in Manhattan, Kansas and will be considered as a Building Occupancy Category II. Figures 6.1 through 6.1 show the plan view, elevations and wall sections.

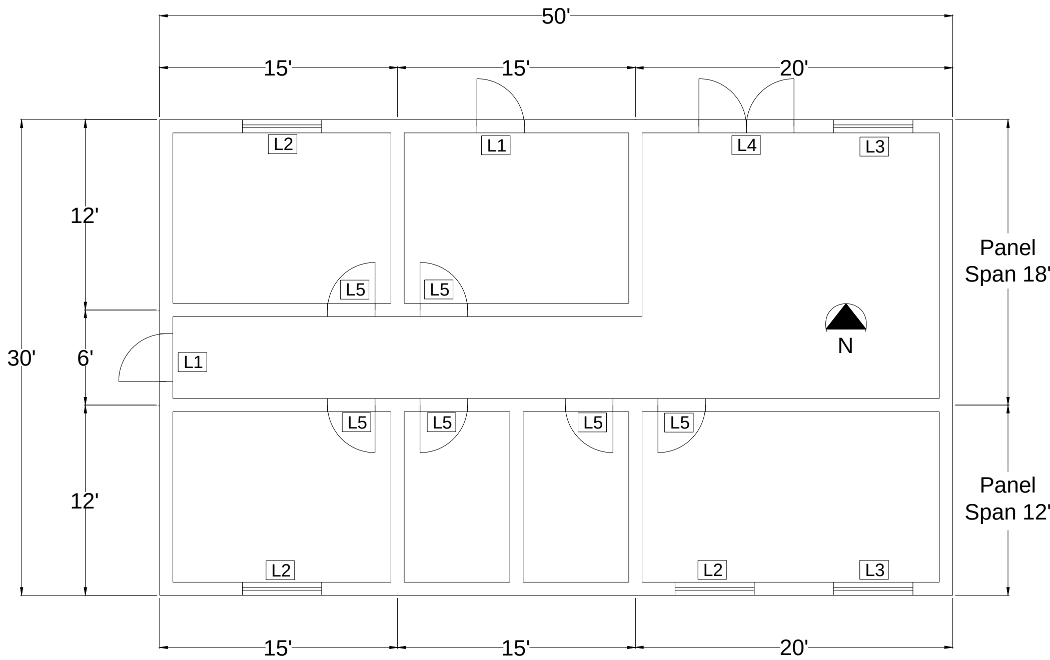


Figure 6.1: Plan View Example Building

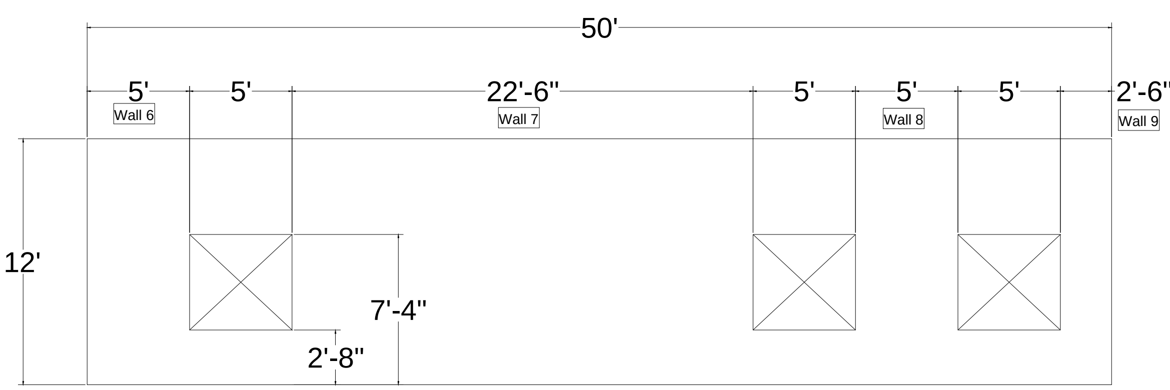


Figure 6.2: South Elevation

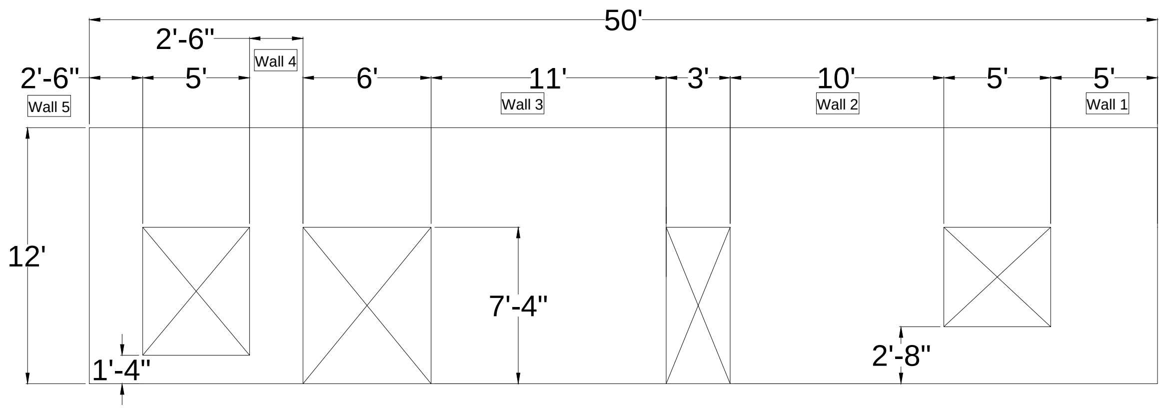


Figure 6.3: North Elevation

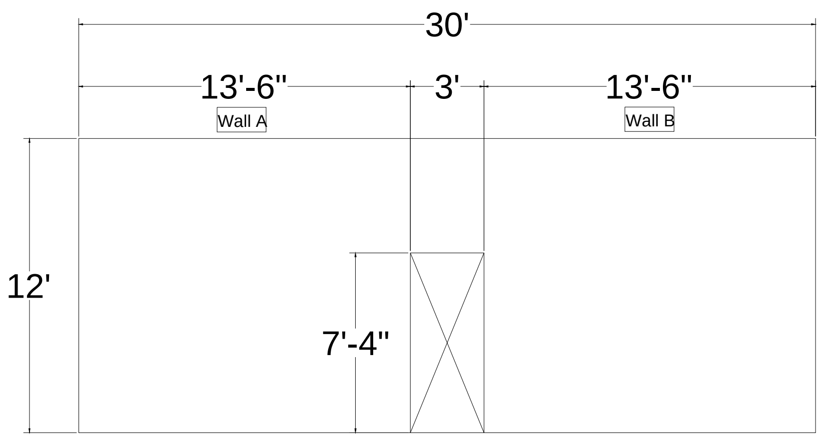


Figure 6.4: West Elevation

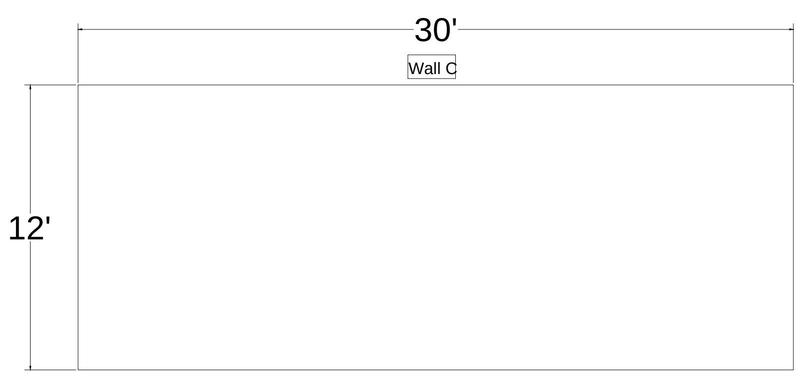


Figure 6.5: East Elevation

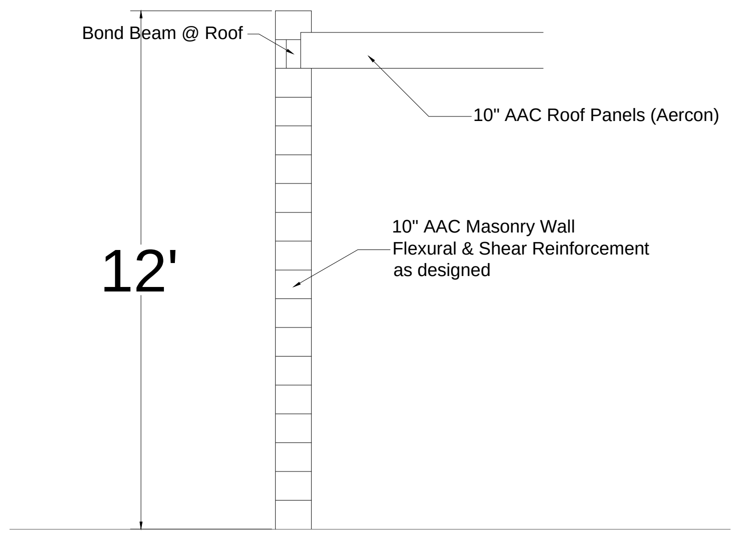


Figure 6.6: Wall Section

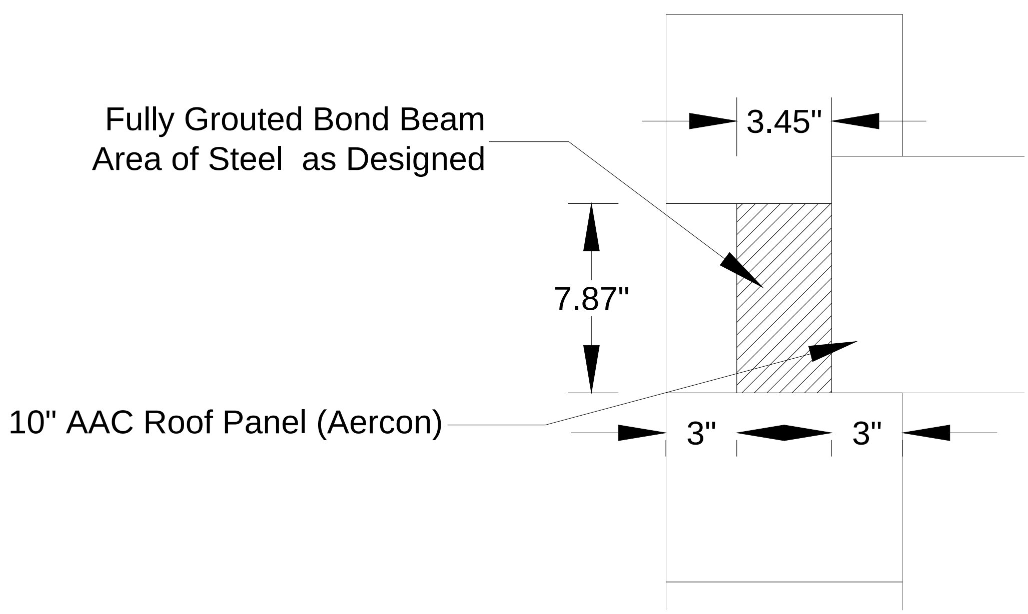


Figure 6.7: Bond Beam at Roof

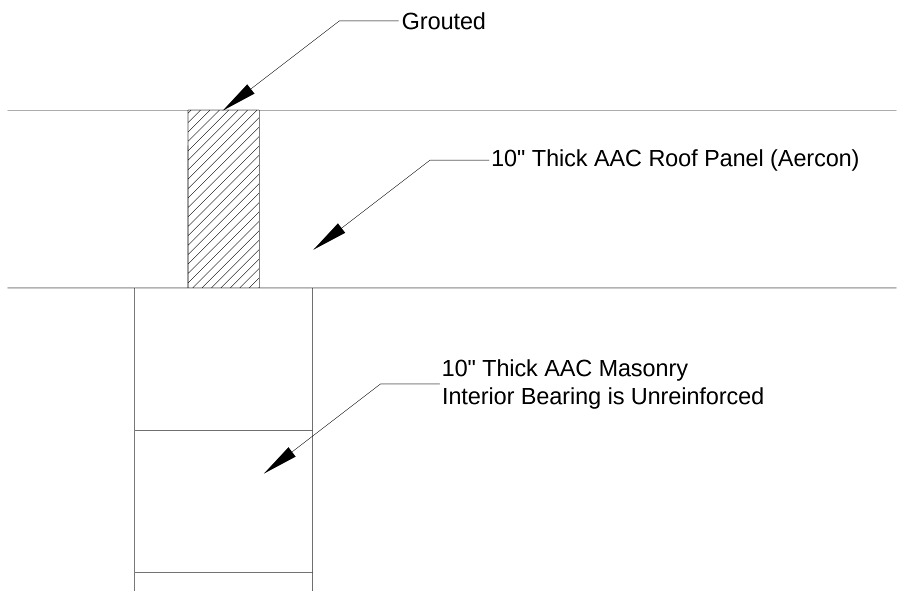


Figure 6.8: Section at Roof Bearing

6.1 Determination of Building Design Loads

The design loads for the example building follow the guidelines found in the ASCE 7-05 Minimum Design Loads for Buildings and Other Structures (ASCE 7-05).

6.1.1 Dead Loads

Dead Load for Roof:

- A 10 inch AAC Roof Panel is used in the construction of this building.
- Roof Panels of length 12 feet and 18 feet are used to span the 30 foot building.
- Strength Class PAAC 4 from ASTM C 1386 is selected as the averaged value.
- The unit weight of each panel is 39 pcf (Aercon)
- Miscellaneous weight is from any extra dead loads on the roof as well as to cover any unknowns.

$$D_{roof} = panel\left(psf\right) + miscellaneous\left(psf\right)$$

$$D_{roof} = 39\,pcf\left(\frac{10in}{12\,{}^{in}\!/_{ft}}\right) + 15\,psf = 47.5\,psf$$

Dead Load for Wall:

- Dead load for wall is to be based on PAAC 4 of ASTM C 1386 for the same reason as chosen in the roof dead load.
- Wall unit weight is 37 pcf ($f'_{AAC} = 580\,psi$)

6.1.2 Live Loads

The building being designed is has only one story, therefore only the roof live load will be considered. The design roof live load is based on ASCE 7-05 Section 4.9.1.

Roof Live Load:

$$L_r = L_o R_1 R_2$$
$$\Rightarrow 12\,psf \le L_r \le 20\,psf$$
(ASCE 7-05 eq. 4-2)

$L_o = 20\,psf$ (ASCE 7-05 t. 4-1)

$R_1 = 1.0$ (Based on tributary area of roof panel maximum 36 sf (2 ft x 18 ft))

$R_2 = 1.0$ (Based on no slope)

$$L_r = 20\,psf\,(1.0)(1.0) = 20\,psf$$

6.1.3 Snow Loads

The building design snow load is based on ASCE 7-05 Chapter 7.

- Building located in Manhattan, Kansas.
- Ground snow load, $p_g = 20\,psf$
- Importance factor is based on Building Occupancy Category II

Flat Roof Snow Load:

$$p_f = 0.7 C_e C_t I p_g$$

$C_e = 1.0$ (Assume partially exposed B)

$C_t = 1.0$ (All other structures)

$I = 1.0$

$$p_f = 0.7(1.0)(1.0)(1.0)(20\,psf) = 14\,psf$$

Check $p_{f\,\min} = I p_g$ for areas where $p_g \le 20\,psf$

$$p_{f\,\min} = (1.0)20\,psf = 20\,psf$$
$$\Rightarrow 14\,psf \le 20\,psf \Rightarrow p_f = 20\,psf$$

6.1.4 Wind Loads

The building design wind loads for the Main Wind Force Resisting System (MWFRS) are used to determine the controlling loads, wind or seismic, of the base shear and are from ASCE 7-05, Section 6.5. The building component design for out of plane wind loads are based on loads determined from ASCE 7-05 Section 6.5 for Components and Cladding.

The building enclosure type will be determined through ASCE 7-05 Section 6.2.

Building Enclosure:

North Wall

- Gross Area, $A_g = 12\,ft\left(50\,ft\right) = 600sf$
- Area of Openings, $A_{op} = 2\left(5\,ft\right)\left(3.34\,ft\right) + 6\,ft\left(7.34\,ft\right) + 5\,ft\left(6\,ft\right) = 107.44sf$

South Wall

- $A_g = 12\,ft\left(50\,ft\right) = 600sf$
- $A_{op} = 3\left(5\,ft\right)\left(3.34\,ft\right) = 50.1sf$

East Wall

- $A_g = 12\,ft\left(30\,ft\right) = 360sf$
- $A_{op} = 0sf$

West Wall

- $A_g = 12\,ft\left(30\,ft\right) = 360sf$
- $A_{op} = 3\,ft\left(7.34\,ft\right) = 22.02sf$

Check for Open Enclosure

$A_{op(1wall)} \geq 0.8A_{g(wall\ considered)}$

No wall satisfies this condition; therefore the building is not open.

Check for Partial Enclosure (if both of the following conditions occur in a wall, the building is Partially Enclosed)

$A_{op(1wall)} > 1.10\sum A_{op(remainder)}$

- North Wall, $107.44sf > 1.10\left(50.1sf + 0sf + 22.02sf\right) = 79.3sf$

$$A_{op(1wall)} > \min\left[4sf, 0.01\left(A_{g(wall\ considered)}\right)\right]$$

$$\frac{A_{op(all)}}{A_{gi}} \leq 0.20$$

- North Wall, $107.44sf > 4sf$
- $\dfrac{A_{op(all)}}{A_{gi}} = \dfrac{179.56sf}{1960sf} = 0.094 \leq 0.20 \Rightarrow O.K.$

The north wall was chosen to be checked because it contains the largest area of openings.

North wall fulfills requirements for partial enclosure.

Building is considered Partially Enclosed because the North Wall met the requirements above for a partially enclosed building. The Analytical Method of ASCE 7-05 Section 6.5 is to be used for the design of wind loads.

Determine MWFRS wind force for base shear calculation:

- Basic Wind Speed, $V = 90mph$ (ASCE 7-05 Fig 6-1)
- $k_d = 0.85$ (to be used in load calculations only) (ASCE 7-05 Tbl 6-4)
- Exposure B, $k_z = 0.70$ (for Case 1) $h = 12\,ft$ (ASCE 7-05 Tbl. 6-3)
- $k_{zt} = 1.0$ (ASCE 7-05 6.5.7.2)
- GC_{pf} with $\theta = 0^o$ (ASCE 7-05 Fig 6-10)

Table 6.1: GC_{pf} for each building surface (ASCE 7-05)

	1	2	3	4	5	6	1E	2E	3E	4E
GC_{pf}	0.4	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43

- $GC_{pi} = \pm 0.55$ (Partially Enclosed) (ASCE 7-05 Fig 6-5)
- $q_h = 0.00256 k_z k_{zt} k_d V^2 I$ (ASCE 7-05 eq.6-15)

  $q_h = 0.00256(0.7)(1.0)k_d(90)^2(1.0) = k_d(14.52\,psf)$
- For Low Rise Buildings

  $p = q_h\left[GC_{pf} - GC_{pi}\right]$ (ASCE 7-05 eq. 6-18)

Table 6.2: Calculation of p (psf) (ASCE 7-05 eq.6-18)

GC_{pi}	1	2	3	4	5	6	1E	2E	3E	4E
+	-2.2	-18.0	-13.4	-12.2	-14.5	-14.5	0.9	-23.5	-15.7	-14.2
-	13.8	-2.0	2.6	3.8	1.5	1.5	16.8	-7.6	0.3	1.7

*Note: positive values signify “towards the surface” negative values signify “away from surface”

Calculation of “a”:

The distance “a” is 10 percent the least horizontal dimension (30 feet) or 40 percent of the mean building height (12 feet) whichever is smaller. The distance “a” is greater than 4 percent of the least horizontal dimension or 3 feet.

$$a = \min\left[0.10(30\,ft) = 3\,ft, 0.4(12\,ft) = 4.8\,ft\right]$$
$$\Rightarrow a = 3\,ft$$

Calculating the base shear:

The worst case scenario in the transverse and longitudinal direction result in a combination of zones 1 (-) plus 4 (+) and zones 1E (-) plus 4E (+). Wall height considered is only half (6 feet) because the assumption is that the remainder of the force is resisted by the slab on grade.

Transverse Base Shear (see Figure below for reference):

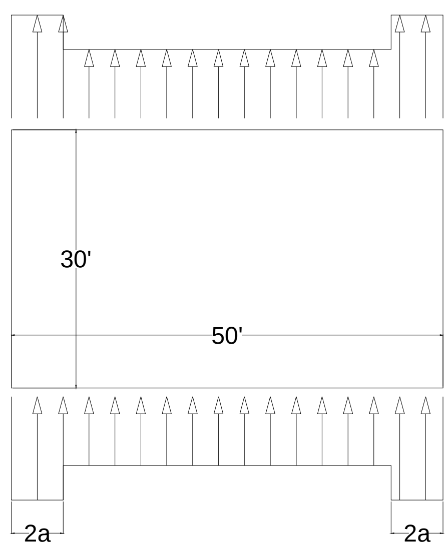


Figure 6.9: Wind in Transverse Direction

$$w_T = (16.8\,psf + 14.2\,psf)(2)(6\,ft)(6\,ft) + (13.8\,psf + 12.2\,psf)(50\,ft - 2(6\,ft))(6\,ft) = 8160\,lbs$$

Longitudinal Base Shear (see figure below for reference):

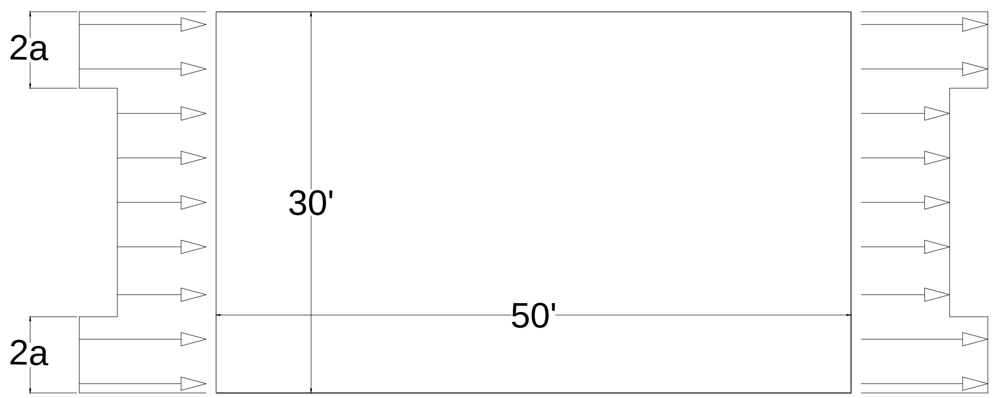


Figure 6.10: Wind in Longitudinal Direction

$$w_L = (16.8\,psf + 14.2\,psf)(2)(6\,ft)(6\,ft) + (13.8\,psf + 12.2\,psf)(30\,ft - 2(6\,ft))(6\,ft) = 5040\,lbs$$

Out of Plane Design Wind Load for Components and Cladding:

The design loads for out of plane wind are determined using ASCE 7-05, Table 6-11a for the values of GC_p. The effective wind area need not be less than one third the wall height multiplied by the wall height. The values found in ASCE 7-05 Table 6-11a are interpolated for the effective area.

Effective Area:

$$A_{eff} = \left(\frac{1}{3}h\right)(h) = \left(\frac{12\,ft}{3}\right)(12\,ft) = 48sf$$

GC_p interpolated values:

For Zone 4 (wall interior) and 5 (wall ends):

$$\left[1 - \frac{48sf - 10sf}{500sf - 10sf}(1.0 - 0.7)\right] = +0.98$$

For Zone 4 (wall interior):

$$\left[-1.1 - \frac{48sf - 10sf}{500sf - 10sf}(-1.1 - (-0.8))\right] = -1.08$$

For Zone 5 (wall ends):

$$\left[-1.4-\frac{48sf-10sf}{500sf-10sf}\left(-1.4-\left(-0.8\right)\right)\right]=-1.35$$

Calculate p for out of plane wind loads:

The out of plane wind loads can be determined by using ASCE 7-05 eq. 6-22 for Low Rise Buildings.

$$p=q_h\left[GC_p-GC_{pi}\right] \qquad \text{(ASCE 7-05 eq. 6-22)}$$

Table 6.3: Calculation of *p* (psf) for out of plane wind load (ASCE 7-05 eq. 6-22)

GC_{pi}	Zone 4 & 5	Zone 4	Zone 5
+	6.2	-23.7	-27.6
-	22.2	-7.7	-11.6

6.1.5 Seismic Loads

The determination of seismic loads is based on ASCE 7-05 Chapters 11 and 12. The base shears calculated in this section are compared with wind to determine which load governs and is therefore be used to determine lateral forces for design.

Determining the Seismic Response Coefficient C_s :

$$S_s=0.206$$
$$S_1=0.053 \qquad \text{(www.usgs.gov) (using zip code 66503)}$$

Soil Class D (No Soil Report)

$$F_a=1.6$$
$$F_v=2.4$$

$$S_{ms}=F_aS_s=1.6\left(0.206\right)=0.3296 \qquad \text{(ASCE 7-05 eq. 11.4-1)}$$

$$S_{m1}=F_vS_1=2.4\left(0.053\right)=0.1272 \qquad \text{(ASCE 7-05 eq. 11.4-2)}$$

$$S_{DS}=\frac{2}{3}S_{ms}=\frac{2}{3}\left(0.3296\right)=0.2197 \qquad \text{(ASCE 7-05 eq. 11.4-3)}$$

$$S_{D1}=\frac{2}{3}S_{m1}=\frac{2}{3}\left(0.1272\right)=0.0848 \qquad \text{(ASCE 7-05 eq. 11.4-4)}$$

Building Occupancy Category II (Importance Factor $I = 1.0$)

Determine Seismic Design Category (SDC):

$$0.167 \le S_{DS} \le 0.33 \Rightarrow SDC\ B$$

$$0.067 \le S_{D1} \le 0.133 \Rightarrow SDC\ B$$

$$T = C_t h_n^x = 0.02(12\,ft)^{0.75} = 0.1289$$

$$T_L = 12$$

$$R = 3 \qquad \text{(Varela 2006)}$$

The value for the seismic force reduction factor (*R*) was obtained through laboratory testing of shear wall specimens by Varela, Tanner, and Kligner (2006).

$$T \le T_L$$

$$\Rightarrow C_{s\max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0848}{0.1289\left(\frac{3}{1.0}\right)} = 0.2193 \qquad \text{(ASCE 7-05 eq.12.8-3)}$$

$$C_{s\min} = 0.01 \qquad \text{(ASCE 7-05 eq. 12.8-5)}$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.2197}{\left(\frac{3}{1.0}\right)} = 0.0732$$

Determining Weight of Building W:

The weight of building will be based on the dead load of the roof plus upper half of the walls and subtracting out the openings. The wall weights will be based on the use of 10 inch AAC masonry.

$$W = 37\,pcf\left(\frac{10in}{12\,{}^{in}\!/_{ft}}\right)\Big[\big(2(30\,ft)+2(50\,ft)\big)(6\,ft)-6(1.34\,ft)(5\,ft)-(1.34\,ft)(3\,ft)-(1.34\,ft)(6\,ft)\Big]$$
$$+47.5\,psf\,(30\,ft)(50\,ft) = 99238.65lbs$$

Determine Seismic Base Shear:

$$V = C_s W = 0.0732(99238.65lbs) = 7264lbs \qquad \text{(ASCE 7-05 eq. 12.8-1)}$$

6.1.6 Base Shear Comparison

Base Shears for wind and seismic are compared to determine which governs wall in-plane shear loads and roof diaphragm forces. See Table 6.4 for a comparison of the wind and seismic base shears. A value of 1.6 is multiplied to wind to account for the load combination differences between Wind and Seismic forces.

Table 6.4: Governing Base Shear

Direction	Wind (1.6W)	Seismic (1.0E)	Governing Case
Transverse	$1.6(8160lbs) = 13056lbs$	7264 lbs	Wind Governs
Longitudinal	$1.6(5040lbs) = 8064lbs$	7264 lbs	Wind Governs

6.1.7 Distribution of Shear forces

The distribution of the shear forces as governed by the wind load are distributed according to the rigidity of each wall section uninterrupted by wall openings. The rigidity of each wall is based on this calculation:

$$R_c = \frac{E_{AAC} t_{eff}}{V} \left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right) \right]^{-1} \qquad \text{(Equation 6.1)}$$

$h = wall\ height$

$L = length\ of\ wall$

This equation is used because the wall is considered cantilevered, fixed at the bottom and free at the top. The value $\frac{E_{AAC} t_{eff}}{V}$ is constant in the direction being considered, i.e. the value would be different when considering longitudinal and transverse directions together and the values of V in either case are different. Since the value is constant in one direction, $\frac{E_{AAC} t_{eff}}{V}$ can be taken out because it will be canceled during calculations.

The value of shear that is applied to a wall is the combination of the direct shear and the shear produced by torsion:

Direct shear ~ $F_v = \frac{R}{\sum R} V$ (Equation 6.2)

Torsion Shear ~ $F_T = \frac{Rd}{\sum Rd^2} M_T$ (Equation 6.3)

The torsion shear comes from the fact that the roof panels perform as a rigid diaphragm. The rigidity of each wall can be found in the following tables using equation 6.1. Refer to the elevations Figures 6.2 through 6.5 for identification of wall marks.

Table 6.5: Transverse Wall Rigidity

Wall	h (ft)	L (ft)	Rigidity
A	12	13.5	0.1826
B	12	13.5	0.1826
C	12	30	0.6868

Table 6.6: Longitudinal Wall Rigidity

Wall	h (ft)	L (ft)	Rigidity
1	12	5	0.016
2	12	10	0.0951
3	12	11	0.1181
4	12	2.5	0.0022
5	12	2.5	0.0022
6	12	5	0.016
7	12	22.5	0.4531
8	12	5	0.016
9	12	2.5	0.0022

Using these rigidities the Center of Rigidity can be found in the “x” and “y” directions. The origin of the axis is taken as the southwest exterior corner of the building. The center of rigidity is found by:

$$\bar{d}_{cr} = \frac{\sum Rd}{\sum R} \qquad \text{(Equation 6.4)}$$

The following tables are the values used in the determination of the center of rigidity.

Table 6.7: Determination of center of rigidity (x-axis)

Wall	R	d (x ft)	R*x
A	0.1826	0.42	0.0767
B	0.1826	0.42	0.0767
C	0.6868	29.58	34.0515

Center of Rigidity (x-axis):

$$\bar{x}_{cr} = \frac{\sum Rx}{\sum R} = \frac{34.2409}{1.052} = 32.5142\,ft$$

Table 6.8: Determination of the center of rigidity (y-axis)

Wall	R	d (y ft)	R*y
1	0.016	29.58	0.4733
2	0.0951	29.58	2.8131
3	0.1181	29.58	3.4934
4	0.0022	29.58	0.0651
5	0.0022	29.58	0.0651
6	0.016	0.42	0.0067
7	0.4531	0.42	0.1903
8	0.016	0.42	0.0067
9	0.0022	0.42	0.0009

Center of Rigidity (y-axis):

$$\bar{y}_{cr} = \frac{\sum Ry}{\sum R} = \frac{7.1146}{0.7209} = 9.8691\,ft$$

The center for rigidity is used to find the eccentricity (in the x and y direction) such that the moment M_T is calculated.

$$e_x = 32.5142\,ft - 25\,ft = 7.5142\,ft$$

$$e_y = 15\,ft - 9.8691\,ft = 5.1309\,ft$$

The moments for each direction of shear can be calculated using the equation:

$$M_T = Ve$$

For Transverse Direction:

$$M_{T_T} = 0.85(8160lbs)(7.5142\,ft) = 52118.5lb\cdot ft$$

For Longitudinal Direction:

$$M_{T_L} = 0.85(5040lbs)(5.1309\,ft) = 21980.8lb\cdot ft$$

Where 0.85 is the k_d factor for wind.

The shear force is distributed to each of the walls following equations 6.2 and 6.3. The proceeding tables show the distribution of direct and torsion shears as well as the total shear that is applied to each wall section.

Table 6.9: Distribution of Shear in the Transverse Direction

Wall	R	x ft(to CR)	R*x	R*x^2	F_V (lbs)	F_T (lbs)	Total (lbs)
A	0.1826	32.0975	5.8610	188.1236	1205	530	1735
B	0.1826	32.0975	5.8610	188.1236	1205	530	1735
C	0.6868	17.0691	11.7231	200.1020	4529	1061	5590

Table 6.10: Distribution of Shear in the Longitudinal Direction

Wall	R	y ft(to CR)	R*y	R*y^2	F_V (lbs)	F_T (lbs)	Total (lbs)
1	0.016	19.7142	0.3154	6.2184	153	83	236
2	0.0951	19.7142	1.8748	36.9606	904	492	1396
3	0.1181	19.7142	2.3282	45.8995	1123	610	1733
4	0.0022	19.7142	0.0434	0.8550	22	12	34
5	0.0022	19.7142	0.0434	0.8550	22	12	34
6	0.016	9.4524	0.1512	1.4296	153	83	236
7	0.4531	9.4524	4.2829	40.4835	4309	1122	5431
8	0.016	9.4524	0.1512	1.4296	153	83	236
9	0.0022	9.4524	0.0208	0.1966	22	12	34

6.2 Design of AAC Components for the Example Building

The example building will be designed using the loads determined in section 6.1. The design requirements that were provided in chapter 5 will be used. The design of all walls, lintels and bond beams will use:

- 10 inch thick AAC masonry units, 8 inch high by 24 inch long.
- Roof panels are 2 feet wide spanning 18 feet and 12 feet.
- Actual dimensions of the wall units are 7.87 inches high x 9.45 inches wide (Aercon)
- Thin-bed mortar is used in bed and head joints
- $f'_{AAC} = 580\,psi$
- $f'_{g} = 2000\,psi$
- $f_y = 60ksi$

6.2.1 North Side Wall Design

Check for Out of Plane Bending and Compression:

Moment of Inertia:

$$I = \frac{12in\left(9.45in\right)^3}{12} = 843.9\, {in^4}/{ft}$$

Area:

$$A = 12in\left(9.45in\right) = 113.4\, {in^2}/{ft}$$

Radius of Gyration:

$$r = \sqrt{\frac{843.9in^4}{113.4in^2}} = 2.73in$$

Slenderness Ratio:

$$\frac{h}{r} = \frac{12\,ft\left(12\, {in}/{ft}\right)}{2.73in} = 52.75$$

Consider the load case $1.2D + 1.6W + 0.5S$ (high moment and large axial load):

$$D = \frac{37\,pcf}{2}(12\,ft)\left(\frac{10in}{12\,{}^{in}\!/_{ft}}\right) + 47.5\,psf\,(9\,ft)$$

$$D = 185\,plf + 427.5\,plf$$

$$S = 20\,psf\,(9\,ft) = 180\,plf$$

$$W = 0.85(27.6\,psf)(1\,ft) = 23.46\,plf$$

Where the 0.85 used in the wind load is k_d.

$$e = \left(\frac{10in}{2} - \frac{3in}{2}\right) = 3.5in$$

The maximum moment can be found at the mid-height of the wall as shown in Figure 6.11.

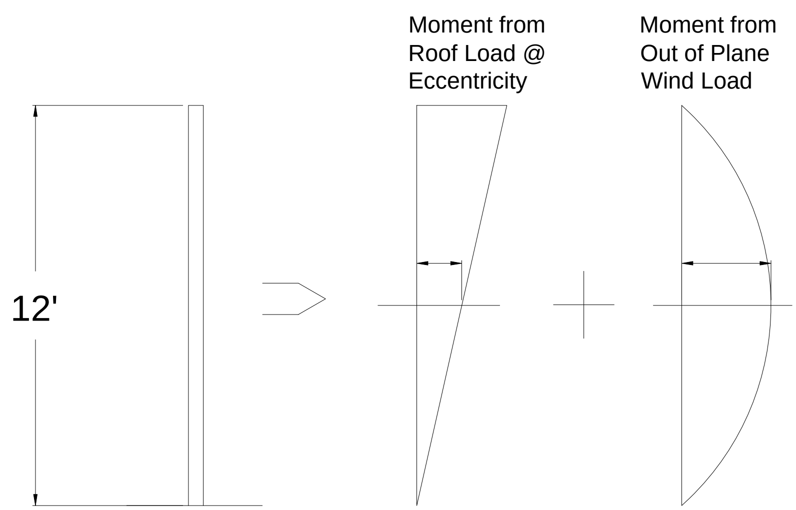


Figure 6.11: Maximum Moment on a Wall

Moment at mid-height of wall from load case:

$$M_u = \frac{[1.2(427.5\,plf) + 0.5(180\,plf)]3.5in}{2} + \frac{1.6(23.46\,plf)(12\,ft)^2}{8}\left(12\,{}^{in}\!/_{ft}\right) = 9163\,{}^{lb\cdot in}\!/_{ft}$$

Axial from load case:

$$P_u = 1.2(185\,plf + 427.5\,plf) + 0.5(180\,plf) = 825\,plf$$

Check equation 5.3.2

$$\frac{P_u}{A_g} = \frac{825\,plf}{113.4\,{}^{in^2}\!/_{ft}} = 7.28\,psi \le 0.2\,f'_{AAC} = 0.2(580\,psi) = 116\,psi \Rightarrow O.K.$$

Axial Strength:

$\frac{h}{r} \leq 99$ Use equation 5.1.4.

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

Steel is not considered because it is not tied (this is the same as using Equation 5.1.2).

$$P_n = 0.80\left[0.85(580\,psi)\left(113.4\,{}^{in^2}\!/_{ft}\right)\right]\left[1 - \left(\frac{144in}{140(2.73in)}\right)^2\right] = 38376\,{}^{lb}\!/_{ft}$$

$$\phi P_n = 0.90\left(38376\,{}^{lbs}\!/_{ft}\right) = 34538.4\,plf \geq P_u = 825\,plf \Rightarrow O.K.$$

Flexural Strength:

P-Delta effect is to be considered to determine the area of steel required. Using section 5.5 Drift Limitations the P-Delta effect can be added to the moment.

P-Delta Effect:

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right) \qquad \text{(Equation 5.5.2)}$$

$$S_n = \frac{I}{y}$$

$$f_{rAAC} = 80\,psi \qquad \text{(Thin-bed Mortar)}$$

$$M_{cr} = \frac{843.9\,{}^{in^4}\!/_{ft}}{\left(\frac{9.45in}{2}\right)}\left(80\,psi + \frac{825\,plf}{113.4\,{}^{in^2}\!/_{ft}}\right) = 15587.6\,{}^{lb \cdot in}\!/_{ft}$$

$$M_u = 9163\,{}^{lb \cdot in}\!/_{ft} < M_{cr} = 15587.6\,{}^{lb \cdot in}\!/_{ft}$$

$$\Rightarrow \delta_u = \frac{5 M_u h^2}{48 E_{AAC} I_g} \qquad \text{(Equation 5.5.3)}$$

$$\delta_u = \frac{5\left(9163\,{}^{lb \cdot in}\!/_{ft}\right)(144in)^2}{48\left(6500(580\,psi)^{0.6}\right)843.9\,{}^{in^4}\!/_{ft}} = 0.0793in$$

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u \qquad \text{(Equation 5.3.3)}$$

$$M_u = \frac{1.6(23.46\,plf)(12\,ft)^2}{8}\left(12\,{}^{in}\!/_{ft}\right) + \left[1.2(427.5\,plf) + 0.5(180\,plf)\right]\frac{3.5in}{2}$$

$$+825\,plf\,(0.0793in) = 9228\,{}^{lb \cdot in}\!/_{ft}$$

The moment calculated is less than the calculated cracking moment therefore no steel is required to resist axial loads and out-of-plane bending. The area of steel will be calculated for use in a comparison with a CMU wall under similar loading condition.

Solve for Area of Steel using equations 5.2.1 and 5.2.2:

$$M_n = \left(A_s f_y + P_u\right) \times \left(d - \frac{a}{2}\right) \qquad \text{(Equation 5.2.1)}$$

$$a = \frac{\left(A_s f_y + P_u\right)}{0.85 f'_{AAC}\, b} \qquad \text{(Equation 5.2.2)}$$

$$\phi M_n \geq M_u \Rightarrow M_n \geq \frac{M_u}{\phi}$$

Substitute *a* into equation 5.2.1

$$\frac{M_u}{\phi} = \left(A_s f_y + P_u\right)\left(d - \frac{\left(A_s f_y + P_u\right)}{2\left(0.85 f'_{AAC}\, b\right)}\right)$$

$$0 = A_s\left(f_y d\right) - A_s^2\left(\frac{f_y^{\,2}}{1.7 f'_{AAC}\, b}\right) - A_s\left(\frac{f_y P_u}{1.7 f'_{AAC}\, b}\right) + \left(P_u d\right) - A_s\left(\frac{P_u f_y}{1.7 f'_{AAC}\, b}\right)$$

$$-\left(\frac{P_u^{\,2}}{1.7 f'_{AAC}\, b}\right) - \left(\frac{M_u}{\phi}\right)$$

This can be put into quadratic equation form:

$$A_s = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$a_{quadratic} = \frac{-f_y^{\,2}}{1.7 f'_{AAC}\, b}$$

$$b_{quadratic} = f_y d - \frac{P_u f_y}{0.85 f'_{AAC}\, b}$$

$$c_{quadratic} = P_u d - \frac{P_u^{\,2}}{1.7 f'_{AAC}\, b} - \frac{M_u}{\phi}$$

$$a_{quadratic} = \frac{-60000^2}{1.7(580\,psi)(12in)} = -304259.63$$

$$b_{quadratic} = 60000\,psi\left(\frac{9.45in}{2}\right) - \frac{825\,plf\,(60000\,psi)}{0.85(580\,psi)(12in)} = 275132.86$$

$$c_{quadratic} = 825\,plf\left(\frac{9.45in}{2}\right) - \frac{(825\,plf)^2}{1.7(580\,psi)(12in)} - \frac{9228\,{}^{lb\cdot in}\!/_{ft}}{0.9} = -6412.73$$

$$A_s = \frac{-275132.86 \pm \sqrt{275132.86^2 - 4(-304259.63)(-6412.73)}}{2(-304259.63)}$$

$$A_s = 0.024\,{}^{in^2}\!/_{ft}$$

Using a # 4 bar at 48 inches on center would be more than adequate.

$$A_s = \frac{0.2in^2}{4\,ft} = 0.05\,{}^{in^2}\!/_{ft}$$

Check maximum area of steel:

$$A_{s\max} = \frac{0.85\,f\,'_{AAC}(0.67d)\left(\frac{0.003}{1.5(0.00207)+0.003}\right)b - \frac{P_u}{\phi}}{f_y} \quad \text{(Equation 5.3.4)}$$

$$A_{s\max} = \frac{0.85(580\,psi)\left(0.67\frac{9.45in}{2}\right)\left(\frac{0.003}{1.5(0.00207)+0.003}\right)(12in) - \frac{825\,plf}{0.90}}{60000\,psi} = 0.138\,{}^{in^2}\!/_{ft}$$

Area of steel is below maximum.

Check the nominal moment capacity with # 4 bars every 48 inches:

$$a = \frac{\left(0.2in^2(60000\,psi) + 825\,plf\,(4\,ft)\right)}{0.85(580\,psi)(48in)} = 0.647in$$

$$M_n = \left(0.2in^2(60000\,psi) + 825(4\,ft)\right)\left(\frac{9.45in}{2} - \frac{0.647in}{2}\right) = 67343lb\cdot in$$

$$\phi M_n = 0.9(67343lb\cdot in) = 60608.7lb\cdot in \geq M_u = 4\,ft\left(9228\,{}^{lb\cdot in}\!/_{ft}\right) = 36912lb\cdot in \Rightarrow O.K.$$

Consider load case $0.9D + 1.6W$ (highest moment with lowest axial):

$$P_u = 0.9(185\,plf + 427.5\,plf) = 551.25\,plf$$

$$M_u = \frac{1.6(23.46\,plf)(12\,ft)^2}{8}\left(12\,{}^{in}\!/\!_{ft}\right) + 0.9(427.5\,plf)\frac{3.5in}{2} = 8781.1\,{}^{lb\cdot in}\!/\!_{ft}$$

Check nominal moment capacity with # 4 bars every 48 inches:

$$a = \frac{\left(0.2in^2(60000\,psi) + 551.25\,plf\,(4\,ft)\right)}{0.85(580\,psi)(48in)} = 0.600in$$

$$M_n = \left(0.2in^2(60000\,psi) + 551.25(4\,ft)\right)\left(\frac{9.45in}{2} - \frac{0.600in}{2}\right) = 62857lb\cdot in$$

$$\phi M_n = 0.9(62857lb\cdot in) = 56571.3lb\cdot in \geq M_u = 4\,ft\left(8781.1\,{}^{lb\cdot in}\!/\!_{ft}\right) = 35124.4lb\cdot in \Rightarrow O.K.$$

No reinforcement is required for axial load and out-of-plane bending.

Check out of plane Shear Strength:

Use the load combo $0.9D + 1.6W$ for highest out of plane shear load.

Check if the section is adequate for out of plane shear:

$$V_{uout-of-plane} = \frac{1.6(23.46\,plf)(12\,ft)}{2} = 225.2\,{}^{lbs}\!/\!_{ft}$$

$$V_{AAC} = 0.8\sqrt{f'_{AAC}}\,bd \qquad \text{(Equation 5.4.7)}$$

$$V_{AAC} = 0.8\sqrt{580\,psi}\,(12in)\left(\frac{9.45in}{2}\right) = 1092.4\,{}^{lbs}\!/\!_{ft}$$

$$\frac{M_u}{V_u d_v} = \frac{8781.1\,{}^{lb\cdot in}\!/\!_{ft}}{\frac{551.25\,plf\,(12\,ft)}{2}\left(\frac{9.45in}{2}\right)} = 0.56 \qquad \text{(Equation 5.4.3)}$$

Interpolation between equations 5.4.2 and 5.4.3 is required.

$$V_n \leq \left(6 - \frac{1-0.56}{1-0.25}(6-4)\right)A_n\sqrt{f'_{AAC}}$$

$$V_n \leq \left(6 - \frac{1-0.56}{1-0.25}(6-4)\right)\left(12in(9.45in)\right)\sqrt{580\,psi} = 13181.8\,{}^{lbs}\!/\!_{ft}$$

Consider no shear reinforcement:

$$V_{AAC} = V_n$$

Check $V_n \leq 13181.8\,{}^{lbs}\!/\!_{ft}$

$$V_{AAC} = V_n = 1092.4\,{}^{lbs}\!/_{ft} \le 13181.8\,{}^{lbs}\!/_{ft} \Rightarrow O.K.$$

Check design shear strength out-of-plane:

$$\phi V_n = 0.75\left(1092.4\,{}^{lb}\!/_{ft}\right) = 819.3\,{}^{lbs}\!/_{ft} \ge V_u = 225.2\,{}^{lbs}\!/_{ft} \Rightarrow O.K.$$

Wall is adequate without shear reinforcement for out-of-plane loads.

Check In-Plane Bending:

The equations to check in-plane bending are the same as the equations used in out of plane bending. Consider Wall 3 (L = 11ft), this is the wall with the highest shear load from shear distribution. Use the load combination $0.9D + 1.6W$ for highest bending with lowest axial load.

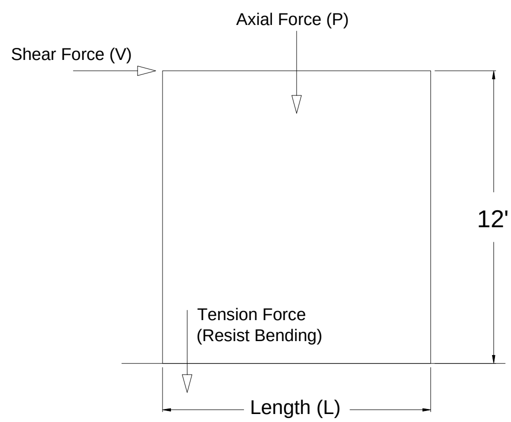


Figure 6.12: In Plane Shear on a wall

$$P_u = 0.9\left(185\,plf + 427.5\,plf\right) = 551.25\,plf$$

$$M_u = V_w h = 1.6\left(1733lbs\right)\left(12\,ft\right) = 33273.6lb \cdot ft$$

Check if # 4 bar at cell at end of wall is adequate:

$$a = \frac{\left(0.2in^2\left(60000\,psi\right) + 551.25\,plf\left(11\,ft\right)\right)}{0.85\left(580\,psi\right)\left(9.45in\right)} = 3.877in$$

$$d = 10.5\,ft - \frac{551.25\,plf\left(11\,ft\right)\left(5\,ft\right)}{0.2in^2\left(60000\,psi\right) + 551.25\,plf\left(11\,ft\right)} = 8.82\,ft$$

$$M_n = \left(0.2in^2\left(60000\,psi\right) + 551.25\left(11\,ft\right)\right)\left(8.82\,ft\left(12\,{}^{in}\!/_{ft}\right) - \frac{3.877in}{2}\right) = 1876851lb \cdot in$$

$$\phi M_n = 0.9(1876851 lb\cdot in)\left(\frac{1}{12 \, in/ft}\right) = 140764 lb\cdot ft \geq M_u = 33273.6 lb\cdot ft \Rightarrow O.K.$$

 # 4 bars at ends of wall are adequate for in plane bending.

Check in plane Shear Strength of wall

Consider Wall # 3 (L = 11ft), this is the wall with the highest shear load from shear distribution. Use the load combo $0.9D + 1.6W$ for highest shear force and lowest axial load on wall.

$$P_u = 0.9(185 plf + 427.5 plf) = 551.25 plf$$

Determine in-plane shear value for V_{AAC} for web shear cracking.

There are mortared head joints, therefore use equation 5.4.4a:

$$V_{AAC} = 0.95 l_w t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}} l_w t}} \qquad \text{(Equation 5.4.4a)}$$

$$l_w = 11 ft\left(12 \, in/ft\right) = 132 in$$

$$t = 9.45 in$$

$$V_{AAC} = 0.95(132 in)(9.45 in)\sqrt{580 psi}\sqrt{1 + \frac{551.25 plf (11 ft)}{2.4\sqrt{580 psi}(132 in)(9.45 in)}} = 29715 lbs$$

Check for crushing of the diagonal strut:

$$\frac{M_u}{V_u d_v} = \frac{V_u h}{V_u d_v} = \frac{h}{d_v} = \frac{12 ft}{10.5 ft} = 1.14 \leq 1.5$$

Crushing of diagonal strut must be considered

$$V_{AAC} = 0.17 f'_{AAC} \, t \frac{h l_w^2}{h^2 + \left(\frac{3}{4} l_w\right)^2} \qquad \text{(Equation 5.4.5)}$$

$$V_{AAC} = 0.17(580 psi)(9.45 in)\frac{144 in (132 in)^2}{(144 in)^2 + \left(\frac{3}{4}(132 in)\right)^2} = 76558 lbs$$

Check sliding shear (thin-bed mortar):

$$V_{AAC} = \mu_{AAC} P_u \qquad \text{(Equation 5.4.6)}$$

$\mu_{AAC} = 1.0$ (Thin-bed mortar)

$V_{AAC} = 1.0\left(551.25\,plf\,(11\,ft)\right) = 6064lbs$

Minimum strength of sliding shear governs governs:

$V_{AAC} = 6064lbs$

Check if shear strength is adequate without shear reinforcement:

$$\frac{M_u}{V_u d_v} = 1.14 \geq 1.00$$

(Equation 5.4.3)

$$\Rightarrow V_n \leq 4A_n\sqrt{f'_{AAC}}$$

$$V_n \leq 4\left(11\,ft\left(12\,{}^{in}\!/\!_{ft}\right)(9.45in)\right)\sqrt{580\,psi} = 120615.5lbs$$

$$V_{AAC} = 6064lbs = V_n \leq 120615.5lbs \Rightarrow O.K.$$

Check design in-plane shear strength:

$$\phi V_n = 0.75(6064lbs) = 4548lbs \geq V_u = 1.6(1733lbs) = 2773lbs \Rightarrow O.K.$$

Wall is adequate without shear reinforcement to resist in plane shear forces.

6.2.2 East Side Wall Design

Check for Out of Plane Bending and Compression:

Section properties are the same as calculated for the North Wall Design

Consider the load case $1.2D + 1.6W + 0.5S$ (high moment and large axial load):

$$D = \frac{37\,pcf}{2}(12\,ft)\left(\frac{10in}{12\,{}^{in}\!/\!_{ft}}\right) + 47.5\,psf\,(1\,ft)$$

$$D = 185\,plf + 47.5\,plf$$

$$S = 20\,psf\,(1\,ft) = 20\,plf$$

$$W = 0.85(27.6\,psf)(1\,ft) = 23.46\,plf$$

*Note: the 0.85 used in the wind load is k_d .

$$e = \left(\frac{10in}{2} - \frac{3in}{2}\right) = 3.5in$$

Moment at mid-height of wall from load case:

$$M_u = \frac{\left[1.2\left(47.5\,plf\right)+0.5\left(20\,plf\right)\right]3.5in}{2} + \frac{1.6\left(23.46\,plf\right)\left(12\,ft\right)^2}{8}\left(12\,{}^{in}\!/_{ft}\right) = 8225\,{}^{lb\cdot in}\!/_{ft}$$

Axial from load case:

$$P_u = 1.2\left(185\,plf + 47.5\,plf\right) + 0.5\left(20\,plf\right) = 289\,plf$$

Check equation 5.3.2

$$\frac{P_u}{A_g} = \frac{289\,plf}{113.4\,{}^{in^2}\!/_{ft}} = 2.55\,psi \le 0.2\,f'_{AAC} = 0.2\left(580\,psi\right) = 116\,psi \Rightarrow O.K.$$

Axial Strength:

$\frac{h}{r} \le 99$ Use equation 5.1.4.

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

Steel is not considered because it is not tied (same as using Equation 5.1.2).

$$P_n = 0.80\left[0.85\left(580\,psi\right)\left(113.4\,{}^{in^2}\!/_{ft}\right)\right]\left[1-\left(\frac{144in}{140\left(2.73in\right)}\right)^2\right] = 38376\,{}^{lb}\!/_{ft}$$

$$\phi P_n = 0.90\left(38376\,{}^{lbs}\!/_{ft}\right) = 34538.4\,plf \ge P_u = 289\,plf \Rightarrow O.K.$$

Flexural Strength:

P-Delta effect is to be considered to determine the area of steel required. Using section 5.5 Drift Limitations the P-Delta effect can be added to the moment.

P-Delta Effect:

$$M_{cr} = S_n\left(f_{rAAC} + \frac{P}{A_n}\right)$$ (Equation 5.5.2)

$$S_n = \frac{I}{y}$$

$$f_{rAAC} = 80\,psi$$ (Thin-bed Mortar)

$$M_{cr} = \frac{843.9\, in^4/ft}{\left(\frac{9.45in}{2}\right)}\left(80\,psi + \frac{289\,plf}{113.4\,in^2/ft}\right) = 14743.4\, lb\cdot in/ft$$

$$M_u = 8225\, lb\cdot in/ft < M_{cr} = 14743.4\, lb\cdot in/ft$$

$$\Rightarrow \delta_u = \frac{5M_u h^2}{48E_{AAC}I_g} \qquad \text{(Equation 5.5.3)}$$

$$\delta_u = \frac{5\left(8225\, lb\cdot in/ft\right)(144in)^2}{48\left(6500(580\,psi)^{0.6}\right)843.9\, in^4/ft} = 0.0712in$$

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u\delta_u \qquad \text{(Equation 5.3.3)}$$

$$M_u = \frac{1.6(23.46\,plf)(12\,ft)^2}{8}\left(12\, in/ft\right) + \left[1.2(47.5\,plf) + 0.5(20\,plf)\right]\frac{3.5in}{2}$$
$$+289\,plf\,(0.0712in) = 8246\, lb\cdot in/ft$$

The moment calculated is less than the cracking moment therefore the wall does not require reinforcement to resist axial loads and out-of-plane bending.

No reinforcement is required for axial load or out-of-plane bending.

Check out of plane Shear Strength:

Use the load combination $0.9D + 1.6W$ for highest out of plane shear load combined with the lowest compressive force.

Check if section is adequate for out-of-plane shear:

$$V_{uout-of-plane} = \frac{1.6(23.46\,plf)(12\,ft)}{2} = 225.2\, lbs/ft$$

$$V_{AAC} = 0.8\sqrt{f'_{AAC}}bd \qquad \text{(Equation 5.4.7)}$$

$$V_{AAC} = 0.8\sqrt{580\,psi}\,(12in)\left(\frac{9.45in}{2}\right) = 1092.4\, lbs/ft$$

$$\frac{M_u}{V_u d_v} = \frac{8183 \, {}^{lb \cdot in}\!/_{ft}}{\frac{209.25\,plf\,(12\,ft)}{2}\left(\frac{9.45in}{2}\right)} = 1.38 \qquad \text{(Equation 5.4.3)}$$

$$\frac{M_u}{V_u d_v} = 1.38 \geq 1.00 \qquad \text{(Equation 5.4.3)}$$

$$\Rightarrow V_n \leq 4A_n\sqrt{f'_{AAC}}$$

$$V_n \leq 4\big(12in(9.45in)\big)\sqrt{580\,psi} = 10924 \, {}^{lbs}\!/_{ft}$$

Consider no shear reinforcement:

$$V_{AAC} = V_n$$

Check $V_n \leq 10924 \, {}^{lbs}\!/_{ft}$

$$V_{AAC} = V_n = 1092.4 \, {}^{lbs}\!/_{ft} \leq 10924 \, {}^{lbs}\!/_{ft} \Rightarrow O.K.$$

Check nominal shear strength out of plane:

$$\phi V_n = 0.75\left(1092.4 \, {}^{lb}\!/_{ft}\right) = 819.3 \, {}^{lbs}\!/_{ft} \geq V_u = 225.2 \, {}^{lbs}\!/_{ft} \Rightarrow O.K.$$

Wall is adequate without shear reinforcement for out of plane loads.

Check In Plane Bending:

The equations to check in plane bending are the same as the equations used in out of plane bending. Consider Wall C (L = 30ft), this is the wall with the highest shear load from shear distribution. Use the load combination $0.9D + 1.6W$ for highest bending with lowest axial load.

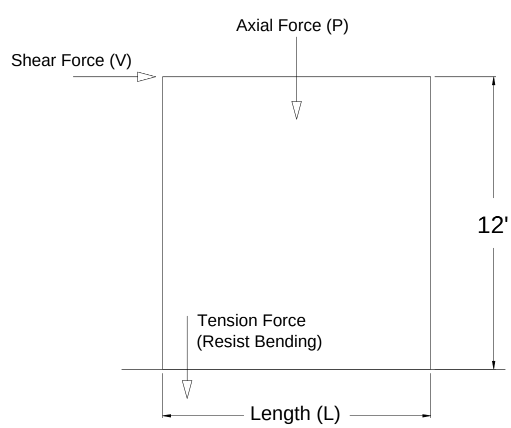


Figure 6.13: In Plane Shear Force on wall

$$P_u = 0.9(185\,plf + 47.5\,plf) = 209.25\,plf$$

$$M_u = V_w h = 1.6(5431lbs)(12\,ft) = 104275.2lb \cdot ft$$

Check if # 4 bar at cell at end of wall is adequate:

$$a = \frac{\left(0.2in^2(60000\,psi) + 209.25\,plf\,(30\,ft)\right)}{0.85(580\,psi)(9.45in)} = 3.923in$$

$$d = 29.5\,ft - \frac{209.25\,plf\,(30\,ft)(14.5\,ft)}{0.2in^2(60000\,psi) + 209.25\,plf\,(30\,ft)} = 24.52\,ft$$

$$M_n = \left(0.2in^2(60000\,psi) + 209.25(30\,ft)\right)\left(24.52\,ft\left(12\,{}^{in}\!/\!_{ft}\right) - \frac{3.923in}{2}\right) = 5342120lb \cdot in$$

$$\phi M_n = 0.9(5342120lb \cdot in)\left(\frac{1}{12\,{}^{in}\!/\!_{ft}}\right) = 400659lb \cdot ft \geq M_u = 104275.2lb \cdot ft \Rightarrow O.K.$$

 # 4 bars at ends of wall are adequate for in plane bending.

Check in plane Shear Strength of wall

Consider Wall C (L = 30ft), this is the wall with the highest shear load from shear distribution. Use the load combo $0.9D + 1.6W$ for highest shear force on wall.

$$P_u = 0.9(185\,plf + 47.5\,plf) = 209.25\,plf$$

Determine in plane shear value for V_{AAC} for web shear cracking.

There are mortared head joints, therefore use equation 5.4.4a:

$$V_{AAC} = 0.95 l_w t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4\sqrt{f'_{AAC}}\, l_w t}} \qquad \text{(Equation 5.4.4a)}$$

$$l_w = 30\,ft\left(12\,{}^{in}\!/\!_{ft}\right) = 360in$$

$$t = 9.45in$$

$$V_{AAC} = 0.95(360in)(9.45in)\sqrt{580\,psi}\sqrt{1 + \frac{209.25\,plf\,(30\,ft)}{2.4\sqrt{580\,psi}\,(360in)(9.45in)}} = 79067lbs$$

Check for crushing of the diagonal strut:

$$\frac{M_u}{V_u d_v} = \frac{V_u h}{V_u d_v} = \frac{h}{d_v} = \frac{12\,ft}{29.5\,ft} = 0.41 \le 1.5$$

Crushing of diagonal strut must be considered

$$V_{AAC} = 0.17 f'_{AAC}\, t \frac{h l_w^2}{h^2 + \left(\frac{3}{4} l_w\right)^2} \qquad \text{(Equation 5.4.5)}$$

$$V_{AAC} = 0.17(580\,psi)(9.45in)\frac{144in(360in)^2}{(144in)^2 + \left(\frac{3}{4}(360in)\right)^2} = 185709lbs$$

Check sliding shear (thin-bed mortar):

$$V_{AAC} = \mu_{AAC} P_u \qquad \text{(Equation 5.4.6)}$$

$$\mu_{AAC} = 1.0 \qquad \text{(Thin-bed mortar)}$$

$$V_{AAC} = 1.0\big(209.25\,plf\,(30\,ft)\big) = 6277.5lbs$$

Minimum strength of sliding shear governs:

$$V_{AAC} = 6277.5lbs$$

Check if shear strength adequate without shear reinforcement:

$$\frac{M_u}{V_u d_v} = 0.41 \le 1.00$$

Interpolation between equations 5.4.2 and 5.4.3 is required.

$$V_n \le \left(6 - \frac{1-0.41}{1-0.25}(6-4)\right) A_n \sqrt{f'_{AAC}}$$

$$V_n \le \left(6 - \frac{1-0.41}{1-0.25}(6-4)\right)\left(30\,ft\left(12\,{}^{in}\!/_{ft}\right)(9.45in)\right)\sqrt{580\,psi} = 362681\,{}^{lbs}\!/_{ft}$$

$$V_{AAC} = 6277.5lbs \ngeq \frac{V_u}{\phi} = \frac{1.6(5431lbs)}{0.75} = 11587lbs$$

Therefore shear reinforcement is required

Determine horizontal steel needed if # 4 bars are used:

$$V_u \le \phi V_n = \phi V_{AAC} + \phi V_s$$

$$\Rightarrow V_s = \frac{V_u}{\phi} - V_{AAC}$$

$$V_s = \frac{1.6(5431 lbs)}{0.75} - 6277.5 lbs = 5309 lbs$$

$$V_s = \frac{A_v}{s} f_y d_v \qquad \text{(Equation 5.4.8)}$$

$$V_s = \frac{A_v}{s} f_y d_v \Rightarrow s = \frac{A_v}{V_s} f_y d_v$$

$$s = \frac{0.20 in^2}{5309 lbs}(60000 psi)(354 in) = 800.2 in$$

Consider a # 4 bar every 6 feet

$$V_s = \frac{0.20}{72 in}(60000 psi)(354 in) = 59000 lbs$$

$$V_n = 6277.5 lbs + 59000 lbs = 65277.5 lbs \le 362681 lbs$$

Check design in-plane shear strength:

$$\phi V_n = 0.75(65277.5 lbs) = 48958 lbs \ge V_u = 1.6(5431 lbs) = 8689.6 lbs \Rightarrow O.K.$$

 # 4 bar every 6 feet for horizontal shear reinforcement to resist in plane shear forces is more than adequate.

6.2.3 Interior Bearing Wall Design

The interior bearing wall is to be designed as an unreinforced AAC wall for axial load only. Consider the load combination $1.2D + 1.6S$. Design as an unreinforced AAC wall although the requirements of MSJC 2005 may require reinforcement to be placed for seismic requirements for this SDC (SDC B) the MSJC 2005 Code does not require reinforcement. This wall will be checked for resisting axial load only then the required steel will be stated at the end of this wall design.

Check for Compression:

Moment of Inertia:

$$I = \frac{12 in(9.45 in)^3}{12} = 843.9 \, in^4/ft$$

Area:

$$A = 12 in(9.45 in) = 113.4 \, in^2/ft$$

Radius of Gyration:

$$r = \sqrt{\frac{843.9in^4}{113.4in^2}} = 2.73in$$

Slenderness Ratio:

$$\frac{h}{r} = \frac{12\,ft\left(12\,{}^{in}\!/\!_{ft}\right)}{2.73in} = 52.75$$

Consider the given load case:

$$D = \frac{37\,pcf}{2}(12\,ft)\left(\frac{10in}{12\,{}^{in}\!/\!_{ft}}\right) + 47.5\,psf\,(15\,ft)$$

$$D = 185\,plf + 712.5\,plf$$

$$S = 20\,psf\,(15\,ft) = 300\,plf$$

$$P_u = 1.2(185\,plf + 712.5\,plf) + 1.6(300\,plf) = 1557\,plf$$

Axial Strength:

$\frac{h}{r} \le 99$ Use equation 5.1.2.

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

$$P_n = 0.80 \times \left[0.85\left(113.4\,{}^{in^2}\!/\!_{ft}\right)(580\,psi)\left(1 - \left(\frac{144in}{140(2.73in)}\right)^2\right)\right] = 38376lbs$$

$$\phi P_n = 0.60\left(38376\,{}^{lbs}\!/\!_{ft}\right) = 23025.6\,plf \ge P_u = 1557\,plf \Rightarrow O.K.$$

Interior bearing wall is adequate as an un-reinforced bearing wall.

For an AAC wall that is not part of the lateral force resisting system for SDC B, the MSJC 2005 Section 1.14.4 does not require any horizontal or vertical reinforcement.

6.2.4 Bond Beam at Roof

A fully grouted bond beam is to surround the building at the roof level as shown in Figure 6.7. The bond beams must be made to withstand forces in the roof diaphragm from wind or seismic loads. The lateral load from the wind or seismic causes tension and compression forces to act on the perimeter of the diaphragm. Imagine that the roof diaphragm acts as a large uniformly loaded beam where the "top" of the beam is affected by compression and the "bottom" is subjected to tension. The ends of this "beam" are collectors that transfer wind shear to the resisting shear wall. The strength of the bond beam in compression is based on the strength of the grout. Depending upon the direction being considered, collectors are to withstand the compression forces produced from the wind whereas the chord forces, the "top" and "bottom" of the "beam," resist the moment produced by the wind. See Figure 6.14 which shows the wind forces. In the case of this building the collectors are not applicable as the shear is transferred directly between the roof diaphragm and the wall. This would not be the case for a wall with a full height opening as a collector would be needed to transfer forces between the roof and wall.

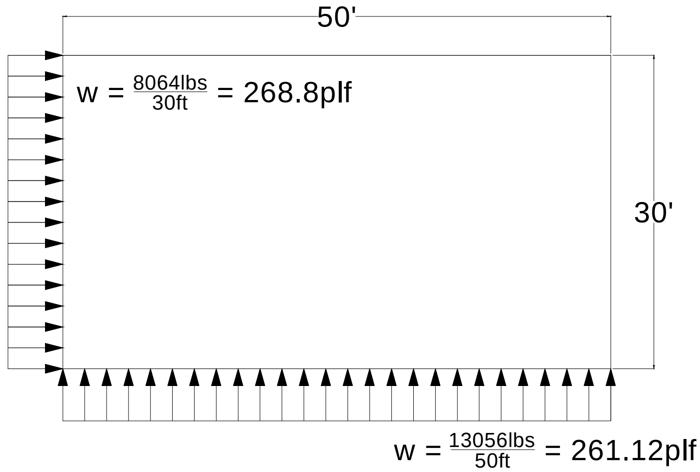


Figure 6.14: Forces on Roof Bond Beam

In the transverse direction (up to down in Figure 6.14):

$$T_{chords} = \frac{261.12\,plf\,(50\,ft)^2}{8\left(30\,ft - \frac{5in}{12\,in/ft}\right)} = 2758lbs$$

In the longitudinal direction (left to right in Figure 6.14):

$$T_{chords} = \frac{268.8\,plf\,(30\,ft)^2}{8\left(50\,ft - \frac{5in}{12\,in/ft}\right)} = 608lbs$$

Design for 50 foot and 30 foot bond beams:

Design the bond beam to resist tension (compression) chord forces.

Assume a # 4 bar to resist tension forces (# 4 bar has commonly been used in the building).

Resistance to tension is calculated as follows for the steel in tension:

Nominal tension strength $= A_s f_y$

$$A_s f_y = 0.2in^2 (60000\,psi) = 12000lbs$$

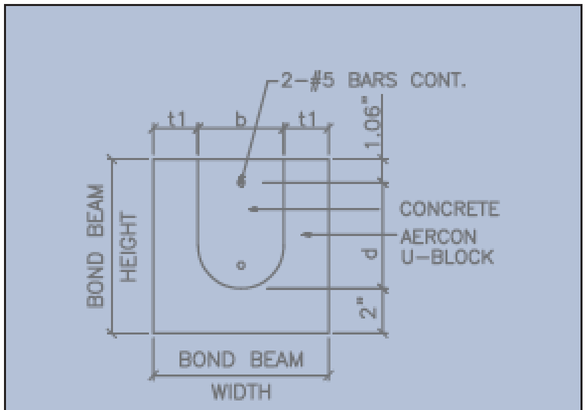
$$\frac{T_{chords}}{\phi} \leq 12000lbs$$

$$\frac{T_{chords}}{\phi} = \frac{2758lbs}{0.9} = 3064.5lbs \leq 12000lbs$$

A # 4 bar is adequate for tension (compression) in both the longitudinal and transverse direction.

6.2.5 Design Lintels for Doors and Windows

The design of lintels to resist moment and shear forces is based on the area and compressive strength of the grout in the AAC bond beam. The area of AAC can be neglected because the compressive strength of AAC is much less than the compressive strength of the grout. The blocks to be used are Aercon U-Blocks, as shown in Figure 6.15 below. The bond beam blocks are the of the same dimension as the standard 8 in x 10 in x 24 in AAC masonry that is being used in the building except that there is a hollowed out core along the length. Data for bond beam is described in Figure 6.15.



NOMINAL WIDTH	t1	b
8"	2"	3.87"
10"	2"	5.45"
12"	3.18"	5.45"

NOMINAL HEIGHT	d
8"	4.81"
9½"	6.39"

Figure 6.15: U-Block for Lintel Design (Aercon)

Consideration is taken into account for the possibility of arching action, a condition where masonry in running bond distributes loads similar to an arch, of the masonry. Several conditions must be fulfilled before arching action can be used:

Condition 1: The height above the opening must be sufficient.

$$h_{above} = 40in \geq \frac{1}{2} width_{opening} + 8in$$

Condition 2: Masonry layout must be in running bond.

Condition 3: There can be no control joint adjacent to lintel.

Condition 4: A minimum bearing of 4 inches is required.

Condition 5: Sufficient strength must be provided to resist lateral thrust.

The 40 inches from condition 1 is the difference between the top of the opening and the bottom of the roof panel bearing elevation. Table 6.11 describes which lintels being designed are subject to arching action based on satisfying the above criteria.

Table 6.11: Determination of Arching Action

Condition	L1 (3ft)	L2 (5ft)	L3 (5ft)	L4 (6ft)	L5 (3ft)
1	Yes (26in)	Yes (38in)	Yes (38 in)	No (44 in)	Yes (26 in)
2	Yes	Yes	Yes	Yes	Yes
3	Yes	Yes	Yes	Yes	Yes
4	Yes	Yes	Yes	Yes	Yes
5	Yes	Yes	No	No	No
Arching?	**Yes**	**Yes**	**No**	**No**	**No**

Design lintel L1:

Lintel L1 has an opening of 3 feet. Therefore it is assumed that two bond beam blocks laid in length are sufficient for this span.

- Lintel Length: 4 ft
- 6-inch bearing each end
- Effective Span: $Span_{effective} = bearing + opening = 0.5\,ft + 3\,ft = 3.5\,ft$
- Bond beam with grout: $w = 39\,plf$ (Aercon U-Block)
- Arching action applies.
- Governing load combination is $1.4D$

Shear on the Lintel:

$$V_u = \frac{w_{lintel}L}{2} + \frac{W_{wall}L}{2}$$

$$W_{wall} = \frac{1}{2}h_{above}t\left(wall(pcf)\right)$$

$$W_{wall} = \frac{1}{2}(26in)(9.45in)(37\,pcf)\frac{1}{144\,in^2/ft^2} = 31.57\,plf$$

$$V_u = \frac{(1.4(39\,plf))(3.5\,ft)}{2} + \frac{1.4(31.57\,plf)3.5\,ft}{2} = 172.9lbs$$

Moment on the Lintel:

$$M_u = \frac{w_{lintel}L^2}{8} + \frac{W_{wall}L^2}{3}$$

$$M_u = \frac{1.4(39\,plf)(3.5\,ft)^2}{8} + \frac{1.4(31.57\,plf)(3.5)^2}{3} = 264.1 lb \cdot ft$$

Check design moment strength:

The equations 5.2.1 and 5.2.2 have been changed to reflect the use of the grout.

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

$$a = \frac{(A_s f_y)}{0.85 f'_g\, b_{grout}}$$

Try a # 4 bar at depth $d = 4in$

$$a = \frac{0.2in^2(60000\,psi)}{0.85(2000\,psi)(5.45in)} = 1.295in$$

$$M_n = 0.2in^2(60000\,psi)\left(4in - \frac{1.295in}{2}\right) = 40230 lb \cdot in$$

$$\phi M_n = 0.9(40230 lb \cdot in)\frac{1}{12 \, in/ft} = 3017.25 lb \cdot ft \geq M_u = 264.1 lb \cdot ft \Rightarrow O.K.$$

One # 4 bar at *d* = 4*in* is adequate for flexure.

Check design shear strength:

$$V_m = \left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_n \sqrt{f'_g}$$

$$\frac{M_u}{V_u d_v} \leq 1.0$$

$$\frac{M_u}{V_u d_v} = \frac{264.1 lb \cdot ft}{172.9 lbs\left(\frac{4in}{12\, in/ft}\right)} = 4.58 \Rightarrow \frac{M_u}{V_u d_v} = 1.0$$

$$V_m = (4.0 - 1.75(1.0))(5.45in(6in))\sqrt{2000\,psi} = 3290.4 lbs$$

$$\phi V_m = 0.75(3290.4 lbs) = 2467.8 lbs \geq V_u = 172.9 lbs \Rightarrow O.K.$$

Shear reinforcement is not required for this lintel.

Design lintel L2:

Lintel L2 has an opening of 5 ft feet. Therefore it is assumed that three bond beam blocks, laid in length, are sufficient for this span.

- Lintel Length: 6 ft
- 6-inch bearing each end
- Effective Span: $Span_{effective} = bearing + opening = 0.5\,ft + 5\,ft = 5.5\,ft$
- Bond beam with grout: $w = 39\,plf$ (Aercon U-Block)
- Arching action applies
- Governing load combination is $1.4D$

Shear on the Lintel:

$$V_u = \frac{w_{l\,\mathrm{int}\,el}L}{2} + \frac{W_{wall}L}{2}$$

$$W_{wall} = \frac{1}{2}h_{above}t\left(wall\left(pcf\right)\right)$$

$$W_{wall} = \frac{1}{2}\left(38in\right)\left(9.45in\right)\left(37\,pcf\right)\frac{1}{144\,in^2/ft^2} = 46.13\,plf$$

$$V_u = \frac{\left(1.4\left(39\,plf\right)\right)\left(5.5\,ft\right)}{2} + \frac{1.4\left(46.13\,plf\right)5.5\,ft}{2} = 327.8lbs$$

Moment on the Lintel:

$$M_u = \frac{w_{l\,\mathrm{int}\,el}L^2}{8} + \frac{W_{wall}L^2}{3}$$

$$M_u = \frac{1.4\left(39\,plf\right)\left(5.5\,ft\right)^2}{8} + \frac{1.4\left(46.13\,plf\right)\left(5.5\right)^2}{3} = 857.7lb\cdot ft$$

Check design moment strength:

The same steel and depth can be applied from Lintel L1 (# 4 bar at d = 4in) to design Lintel L2

$$\phi M_n = 0.9\left(40230lb\cdot in\right)\frac{1}{12\,in/ft} = 3017.25lb\cdot ft \geq M_u = 857.7lb\cdot ft \Rightarrow O.K.$$

 # 4 bar at *d* = 4*in* is adequate for flexure.

Check design shear strength:

$$V_m = \left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_n \sqrt{f'_g}$$

$$\frac{M_u}{V_u d_v} \le 1.0$$

$$\frac{M_u}{V_u d_v} = \frac{857.7 lb \cdot ft}{327.8 lbs\left(\frac{4in}{12 \, in/ft}\right)} = 7.85 \Rightarrow \frac{M_u}{V_u d_v} = 1.0$$

$$V_m = \left(4.0 - 1.75(1.0)\right)\left(5.45in(6in)\right)\sqrt{2000\,psi} = 3290.4lbs$$

$$\phi V_m = 0.75(3290.4lbs) = 2467.8lbs \ge V_u = 327.8lbs \Rightarrow O.K.$$

Shear reinforcement is not required for this lintel.

Design lintel L3:

Lintel L3 has an opening of 5 ft feet. Therefore it is assumed that three bond beam blocks, laid in length, are sufficient for this span.

- Lintel Length: 6 ft
- 6-inch bearing each end
- Effective Span: $Span_{effective} = bearing + opening = 0.5\,ft + 5\,ft = 5.5\,ft$
- Bond beam with grout: $w = 39\,plf$ (Aercon U-Block)
- Arching Action doesn't apply.
- Governing load combo is $1.2D + 1.6S$ (Highest Load)

Shear on the Lintel:

$$V_u = \frac{w_{lintel} L}{2} + \frac{w_{wall} L}{2}$$

$$w_{wall} = Roof + Wall$$

$$W_{wall} = 1.2\left[\left(37\,pcf\,(4\,ft)\left(\frac{10in}{12\,in/ft}\right)\right) + 47.5\,psf\,(9\,ft)\right] + 1.6\left[20\,psf\,(9\,ft)\right] = 949\,plf$$

$$V_u = \frac{\left(1.2(39\,plf)\right)(5.5\,ft)}{2} + \frac{949\,plf\,(5.5\,ft)}{2} = 2738.5lbs$$

Moment on the Lintel:

$$M_u = \frac{w_{l\,\mathrm{int}\,el} L^2}{8} + \frac{w_{wall} L^2}{8}$$

$$M_u = \frac{1.2\left(39\,plf\right)\left(5.5\,ft\right)^2}{8} + \frac{949\,plf\left(5.5\,ft\right)^2}{8} = 3765.4\,lb\cdot ft$$

Check design moment strength:

From before with a # 4 bar:

$$\phi M_n = 3017.25\,lb\cdot ft$$

Not adequate for the applied moment.

Try a # 5 bar at depth $d = 4in$

$$a = \frac{0.31in^2\left(60000\,psi\right)}{0.85\left(2000\,psi\right)\left(5.45in\right)} = 2.008in$$

$$M_n = 0.31in^2\left(60000\,psi\right)\left(4in - \frac{2.008in}{2}\right) = 55725.6\,lb\cdot in$$

$$\phi M_n = 0.9\left(55725.6\,lb\cdot in\right)\frac{1}{12\,in/ft} = 4179.42\,lb\cdot ft \geq M_u = 3765.4\,lb\cdot ft \Rightarrow O.K.$$

 # 5 bar at *d* = 4*in* is adequate for flexure.

Check design shear strength:

$$V_m = \left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_n \sqrt{f'_g}$$

$$\frac{M_u}{V_u d_v} \leq 1.0$$

$$\frac{M_u}{V_u d_v} = \frac{3765.4\,lb\cdot ft}{2738.5\,lbs\left(\frac{4in}{12\,in/ft}\right)} = 4.12 \Rightarrow \frac{M_u}{V_u d_v} = 1.0$$

$$V_m = \left(4.0 - 1.75\left(1.0\right)\right)\left(5.45in\left(6in\right)\right)\sqrt{2000\,psi} = 3290.4\,lbs$$

$$\phi V_m = 0.75\left(3290.4\,lbs\right) = 2467.8\,lbs \ngeq V_u = 2738.5\,lbs \Rightarrow N.G.$$

Shear reinforcement is required for this lintel.

$$V_s = \frac{V_u - \phi V_m}{\phi}$$

$$V_s = \frac{2738.5lbs - 2467.8lbs}{0.75} = 361lbs$$

$$V_s = \frac{A_v}{s} f_y d_v \qquad \text{(Equation 5.4.8)}$$

$$A_v = \frac{sV_s}{f_y d_v}$$

Try prescribed maximum from 2005 MSJC Code Section A.3.4.2.3 of 50 percent beam depth:

$$s = \frac{h}{2} = \frac{7.87in}{2} = 3.935in:$$

$$A_v = \frac{3.935in(361lbs)}{60000psi(4in)} = 0.006in^2$$

Use a # 3 bar every 3.935 inches ($A_v = 0.11in^2$)

Distance where shear reinforcement can end:

$$V_u - x\left(\frac{V_u}{\frac{L}{2}}\right) = \phi V_m \Rightarrow x = \frac{\frac{L}{2}(V_u - \phi V_m)}{V_u}$$

$$x = \frac{\frac{5.5ft}{2}(2738.5lbs - 2467.8lbs)}{2738.5lbs}\left(12\,{}^{in}\!/\!_{ft}\right) = 3.26in$$

Therefore 1 # 3 bar at 1 inch from each end is adequate for shear reinforcement.

Design lintel L4:

Lintel L4 has an opening of 6 ft, 4 bond beam blocks, laid in a row, are sufficient for this span.

- Lintel Length: 8 ft
- 1 foot bearing length.
- Effective Span: $Span_{effective} = bearing + opening = 1\,ft + 6\,ft = 7\,ft$
- Bond beam with grout: $w = 39\,plf$ (Aercon U-Block)

- Arching Action doesn't apply.
- Governing load combo is $1.2D + 1.6S$ (Highest Load)

Shear on the Lintel:

$$V_u = \frac{w_{l\,\text{int}\,el} L}{2} + \frac{w_{wall} L}{2}$$

$$w_{wall} = Roof + Wall$$

$$W_{wall} = 1.2\left[\left(37\,pcf\,(4\,ft)\left(\frac{10in}{12\,in/ft}\right)\right) + 47.5\,psf\,(9\,ft)\right] + 1.6\left[20\,psf\,(9\,ft)\right] = 949\,plf$$

$$V_u = \frac{(1.2(39\,plf))(7\,ft)}{2} + \frac{949\,plf\,(7\,ft)}{2} = 3485.3lbs$$

Moment on the Lintel:

$$M_u = \frac{w_{l\,\text{int}\,el} L^2}{8} + \frac{w_{wall} L^2}{8}$$

$$M_u = \frac{1.2(39\,plf)(7\,ft)^2}{8} + \frac{949\,plf\,(7\,ft)^2}{8} = 6099.3lb \cdot ft$$

Check design moment strength:

From before with a # 5 bar at 4 in:

$$\phi M_n = 4179.42lb \cdot ft$$

Not adequate for the applied moment.

Try a # 6 bar at depth $d = 5in$ (depth available for reinforcement is 5.87 inches, 5 inches is used such that enough space is allowed below the bar)

$$a = \frac{0.44in^2\,(60000\,psi)}{0.85(2000\,psi)(5.45in)} = 2.849in$$

$$M_n = 0.44in^2\,(60000\,psi)\left(5in - \frac{2.849in}{2}\right) = 94393.2lb \cdot in$$

$$\phi M_n = 0.9(94393.2lb \cdot in)\frac{1}{12\,in/ft} = 7079.5lb \cdot ft \geq M_u = 6099.3lb \cdot ft \Rightarrow O.K.$$

 # 6 bar at *d* = 5*in* is adequate for flexure.

Check design shear strength:

$$V_m = \left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_n \sqrt{f'_g}$$

$$\frac{M_u}{V_u d_v} \leq 1.0$$

$$\frac{M_u}{V_u d_v} = \frac{6099.3 lb \cdot ft}{3485.4 lbs\left(\frac{5in}{12 \, in/ft}\right)} = 4.20 \Rightarrow \frac{M_u}{V_u d_v} = 1.0$$

$$V_m = \left(4.0 - 1.75(1.0)\right)\left(5.45in(6in)\right)\sqrt{2000\,psi} = 3290.4lbs$$

$$\phi V_m = 0.75(3290.4lbs) = 2467.8lbs \not\geq V_u = 3485.3lbs \Rightarrow N.G.$$

Shear reinforcement is required for this lintel.

$$V_s = \frac{V_u - \phi V_m}{\phi}$$

$$V_s = \frac{3485.3lbs - 2467.8lbs}{0.75} = 1356.7lbs$$

$$V_s = \frac{A_v}{s} f_y d_v \qquad \text{(Equation 5.4.8)}$$

$$A_v = \frac{sV_s}{f_y d_v}$$

Try prescribed maximum from 2005 MSJC Code Section A.3.4.2.3 of 50 percent beam depth $s = \frac{h}{2} = \frac{7.87in}{2} = 3.935in$:

$$A_v = \frac{3.935in(1356.7lbs)}{60000\,psi(5in)} = 0.018in^2$$

Use a # 3 bar every 3.935 inches ($A_v = 0.11in^2$)

Distance where shear reinforcement can end:

$$V_u - x\left(\frac{V_u}{\frac{L}{2}}\right) = \phi V_m \Rightarrow x = \frac{\frac{L}{2}(V_u - \phi V_m)}{V_u}$$

$$x = \frac{\frac{7\,ft}{2}(3485.3lbs - 2467.8lbs)}{3485.3lbs}\left(12\,{}^{in}\!/_{ft}\right) = 12.26in$$

$$\frac{x}{s} + 1 = \#bars\ required$$

Use an $s = 3.5in$

$$\frac{x}{s} + 1 = \frac{12.26in}{3.5in} + 1 = 5bars$$ (This value is rounded up)

Therefore 1 # 3 bar at 1 inch and 4 # 3 bars at 3.5 inches from each end is adequate for shear reinforcement.

Design lintel L5:

Lintel L5 has an opening of 3 ft, 2 bond beam blocks, laid in a row, are sufficient for this span.

- Lintel Length: 4 ft
- 6 inch bearing length
- Effective Span: $Span_{effective} = bearing + opening = 0.5\,ft + 3\,ft = 3.5\,ft$
- Bond beam: $w = 39\,plf$ (Aercon U-Block)
- Arching Action doesn't apply.
- Governing load combo is $1.2D + 1.6S$ (Highest Load)

Shear on the Lintel:

$$V_u = \frac{w_{l\,\text{int}\,el} L}{2} + \frac{w_{wall} L}{2}$$

$$w_{wall} = Roof + Wall$$

$$W_{wall} = 1.2\left[\left(37\,pcf\,(2.67\,ft)\left(\frac{10in}{12\,{}^{in}\!/_{ft}}\right)\right) + 47.5\,psf\,(15\,ft)\right] + 1.6\left[20\,psf\,(15\,ft)\right] = 1433.8\,plf$$

$$V_u = \frac{(1.2(39\,plf))(3.5\,ft)}{2} + \frac{1433.8\,plf\,(3.5\,ft)}{2} = 2591.1lbs$$

Moment on the Lintel:

$$M_u = \frac{w_{l\,int\,el} L^2}{8} + \frac{w_{wall} L^2}{8}$$

$$M_u = \frac{1.2(39\,plf)(3.5\,ft)^2}{8} + \frac{1433.8\,plf\,(3.5\,ft)^2}{8} = 2267.2lb \cdot ft$$

Check design moment strength:

From before with a # 4 bar:

$$\phi M_n = 0.9(40230lb \cdot in)\frac{1}{12 \, in/ft} = 3017.25lb \cdot ft \geq M_u = 2267.2lb \cdot ft \Rightarrow O.K.$$

 # 4 bar at *d* = 4*in* is adequate for flexure.

Check design shear strength:

$$V_m = \left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_n \sqrt{f'_g}$$

$$\frac{M_u}{V_u d_v} \leq 1.0$$

$$\frac{M_u}{V_u d_v} = \frac{2267.2lb \cdot ft}{2591.1lbs\left(\frac{4in}{12 \, in/ft}\right)} = 2.62 \Rightarrow \frac{M_u}{V_u d_v} = 1.0$$

$$V_m = (4.0 - 1.75(1.0))(5.45in(6in))\sqrt{2000\,psi} = 3290.4lbs$$

$$\phi V_m = 0.75(3290.4lbs) = 2467.8lbs \not\geq V_u = 2591.1lbs \Rightarrow N.G.$$

Shear reinforcement is required for this lintel.

$$V_s = \frac{V_u - \phi V_m}{\phi}$$

$$V_s = \frac{2591.1lbs - 2467.8lbs}{0.75} = 164.4lbs$$

$$V_s = \frac{A_v}{s} f_y d_v \qquad \text{(Equation 5.4.8)}$$

$$A_v = \frac{sV_s}{f_y d_v}$$

Try prescribed maximum from 2005 MSJC Code Section A.3.4.2.3 of 50 percent beam depth $s = \frac{h}{2} = \frac{7.87in}{2} = 3.935in$:

$$A_v = \frac{3.935in(164.4lbs)}{60000psi(4in)} = 0.003in^2$$

Use a # 3 bar every 3.935 inches ($A_v = 0.11in^2$)

Distance where shear reinforcement can end:

$$V_u - x\left(\frac{V_u}{\frac{L}{2}}\right) = \phi V_m \Rightarrow x = \frac{\frac{L}{2}(V_u - \phi V_m)}{V_u}$$

$$x = \frac{\frac{3.5ft}{2}(2591.1lbs - 2467.8lbs)}{2591.1lbs}\left(12 \, in/ft\right) = 1in$$

Therefore 1 # 3 bar at 1 inch from each end is adequate for shear reinforcement.

6.3 Example Building Reinforcement Summary

This section provides a summary of the reinforcement that was calculated in the previous section.

Walls:

North & South walls use 10 inch thick AAC masonry with no reinforcement for axial and out-of-plane bending (1 # 4 bar is to be located at each end at openings and corners). No shear reinforcement is required for these walls for in-plane or out-of-plane shear.

East & West walls use 10 inch thick AAC masonry with no reinforcement for axial and out-of-plane bending (1 # 4 bar is to be located at each end at openings and corners). For shear reinforcement # 4 bars at 72 inches on center (vertical) is required for in-plane shear. No out-of-plane shear reinforcement is required.

The exterior walls called for in this design example are shear walls and must meet the seismic requirements of the 2005 MSJC Code Section 1.14. # 4 bars are provided at least 24 inches of each side of openings and wall ends and horizontal reinforcement at the top and bottom of wall openings extending not less than 24 inches or 40 bar diameters.

Interior Bearing wall uses 10 inch thick AAC masonry and has been designed as an un-reinforced AAC bearing wall. The 2005 MSJC Code does not require reinforcement for components that are not part of the lateral resistance system for SDC B. This only applies to this building example. Refer to the 2005 MSJC Code for other cases.

Bond Beams:

Single course of bond beam is adequate to resist chord forces applied to the building through the roof diaphragm; 1 # 4 bar placed in the fully grouted bond beam is adequate to resist the chord forces. However, for SDC B the 2005 MSJC Code Section 1.14.4.3 requires that 0.4 square inches of reinforcement is to be placed in the bond beam that anchors the roof diaphragm. Therefore 2 # 4 bars are placed in the fully grouted bond beams at the roof level in all exterior walls.

Lintels:

A following is a listing of the calculated reinforcement requirements for the lintels. The reinforcement calculated although works for the design of this project an alternate solution may be to use double angles to resist the tension and shear produced in place of the reinforcement.

L1 Lintels are 1 course fully grouted lintel blocks with 1 # 4 bar at depth 4inches. No shear reinforcement is required.

L2 Lintels are 1 course fully grouted lintel blocks with 1 # 4 bar at depth 4 inches. No shear reinforcement required.

L3 Lintels are 1 course fully grouted lintel blocks with 1 # 5 bar at depth 4 inches. Shear reinforcement is covered by 1 # 3 bar at each end. The shear reinforcement is not necessary due to the fact that 1 inch from each end is over the bearing length of the lintel. For L3 there is no shear reinforcement required.

L4 Lintels are 1 course fully grouted bond beams with 1 # 6 bar at depth 5 inches. Shear reinforcement is covered by 5 # 3 bars at each end.

L5 Lintels 1 course fully grouted bond beams with 1 # 4 bar at depth 4 inches. Shear reinforcement is covered by 1 # 3 bar at each end. The shear reinforcement is not necessary due to the fact that 1 inch from each end is over the bearing length of the lintel. For L5 there is no shear reinforcement required.

CHAPTER 7 - Comparison of AAC with CMU

The 10 inch thick AAC wall from the Example Design Building in Chapter 6 is to be used in comparison with a 10 inch CMU wall. The CMU wall is designed based on the same axial load and bending moment as the AAC wall to give this comparison some validity. The calculations for the AAC wall show the amount of steel for the specified loads is $0.024\, {in^2}/{ft}$.

This area of steel is comparable to $0.017\, {in^2}/{ft}$ required for the CMU wall. The determination of the required vertical steel value for the CMU is found in Appendix A.

Advantages of the light weight of AAC compared to CMU.

As stated before, AAC is a lightweight concrete material. From Table 4.1 the range of weight for the strength classes is 25 pounds per cubic foot to 50 pounds per cubic foot. From the Design Example 37 pounds per cubic foot masonry units were used, with the 12 foot wall height, this translates to:

$$\frac{9.45in}{12\, {in}/{ft}}(12\, ft)(37\, pcf) = 350\, plf$$

In comparison, a CMU wall of the same thickness (10 inches) has a range of 33.5 pounds per linear foot of wall per foot height of wall to 56.5 pounds per linear foot of wall per foot height of wall. These values were taken from the Masonry Designers Guide 5th Edition (MDG-5). Using a middle value from the design guide of 45 pounds per linear foot of wall per foot height of wall, the 12 foot wall translates to:

$$\frac{9.45in}{12\, {in}/{ft}}(12\, ft)(37\, pcf) = 350\, plf$$

In comparison, a CMU wall of the same thickness (10 inches) has a range of 33.5 pounds per linear foot of wall per foot height of wall to 56.5 pounds per linear foot of wall per foot height of wall. These values were taken from the Masonry Designers Guide 5th Edition. Using a middle value from the design guide of 45 pounds per linear foot of wall per foot height of wall, the 12 foot wall translates to:

$$(12\, ft)(45\, plf) = 540\, plf$$

That is more than 50 percent increase in weight. The lighter weight of AAC could possibly mean faster placement of block or faster construction times (Pytlik & Saxena 1992).

The lightweight of AAC has another advantage. The maximum gross weight limit for trucks on Kansas Interstate highways is 80,000 pounds and the maximum legal dimensions of a truck and trailer combination is 8.5 feet wide, 14 feet tall, and 65 feet long (ksrevenue.org). From the Masonry Designer's Guide the average weight of an 8"x10"x16" CMU is 37.5 pounds. To look at it another way:

$$\frac{80,000 lbs}{37.5 \, {}^{lbs}\!/_{CMU}} = 2133.33 \ CMU$$

$$\frac{2133 \ CMU \left(8in \times 10in \times 16in\right)}{{}^{\left(12in\right)^3}\!/_{ft^3}} = 1580 \, ft^3$$

Using the 37 pounds per cubic foot from the design example for AAC:

$$\frac{80,000 lbs}{37 \, pcf} = 2162.2 \, ft^3$$

From this it is clear that a larger amount of AAC can be transported at one time, even taking into account the size limitations. This higher volume per truck means that money can be saved in the transportation of building materials.

Constructability advantages by using AAC rather than CMU.

Normally in construction when a concrete material needs to be shaped it is poured in that shape on the job site, comes pre-cast in that shape, or through the use of special blades it is cut to the desired shape on the job site. ACC is typically pre-cast or pre-cut into the desired shapes in the manufacturing plant. However, AAC can be cut in the field using normal wood working tools such as handsaws, band saws, etc. Figure 7.1 below shows a block of AAC being cut at the job site with a regular band saw. Diamond blades used for cutting concrete and CMU have a greater cost than a common handsaw for wood. This does not even take into account the fact that regular wood drills and drill bits can be used or the fact that simple hand tools can be used to drive nails into AAC for the attachment of other materials (Pytlik & Saxena 1992).

Figure 7.1: AAC cut with band saw (www.pragmaticconstruction.com)

Comparison of material costs between AAC and CMU.

Using the 2006 RSMeans Building Construction Cost Data, the prices for an 8 inch AAC block can be compared to an 8 inch thick CMU wall based on the square foot of wall constructed. The AAC has a material cost of $3.15 per square foot which is higher than the 8 inch reinforced CMU at $2.02 per square foot. If non-reinforced CMU at a cost of $1.93 per square foot is used, the gap becomes even wider. This initial material cost is clearly a disadvantage for AAC.

Comparison of the insulating value of AAC and CMU.

Referring to the R-values of AAC given in Chapter 4, an 8 inch thick panel of AAC has an R of about 11.5 this is based on a density of 25 to 31 pounds per cubic foot. Likewise, the R-value for 8 inch CMU (Normal weight) with vermiculite in the cells has an R-value of 1.5. For lightweight 8 inch CMU with cores filled with vermiculite the R-value is approximately 4.5. From this information, it can be seen that lower density concrete materials seem to have a higher insulating value. AAC has the ability to absorb large amounts of radiant thermal energy which is released or transmitted back at a low rate (Pytlik & Saxena 1997) this helps in insulating one side of an assembly from the other, the better the ability to insulate the greater the R-value. For CMU to even match the value of AAC, 8 inch CMU (lightweight) with vermiculite filled cores would need additional insulation consisting of 1 inch of rigid insulation (R-value is 9) and 2 layers of 5/8th inch gypsum board (R-value approximately 1) for a total R-value of 11.5.

Comparison of in-plane shear strengths of AAC and CMU walls.

The 10 inch thick AAC wall from the Example Design Building is to be used in comparison with a 10 inch CMU wall. The CMU wall is designed based on the same in-plane shear load as the AAC wall. The design strength of AAC was governed by the sliding shear amounting to 4548 pounds. The CMU as determined from one equation (the three situations are not calculated: web shear cracking, crushing of the diagonal strut, and sliding shear) and was found as 54265.5 pounds. This is much higher than the AAC wall value of 4548 pounds. The strength of the AAC for this example is determined from the friction between AAC block and the thin-set mortar that was used. The determination of the in-plane shear strength for the CMU is found in Appendix A.

CHAPTER 8 - Conclusion

Autoclaved Aerated Concrete (AAC), is a product that has been used in construction for more than 80 years beginning in Europe, expanding to other nations and recently to the United States. AAC is a lightweight concrete building material that is cut into masonry units or produced in larger panels or planks. AAC units are used for walls, both load bearing and non-load bearing partitions, and for roof or floor framing using planks. The purpose of this paper was to present the history of AAC, a description of the manufacturing process, a description of the material and performance properties of AAC, and present the design requirements of AAC using strength design. In depth, this report has described the process by which AAC can be designed including example problems as well as a design building example. This report has also shown how AAC can be directly compared to CMU (Table 8.1). The purpose of this report has been to increase the understanding of what AAC is and to provide details of its capabilities as a building material.

Table 8.1: Comparison Summary

Comparison Criteria	AAC	CMU
Rebar Comparison	Greater than to CMU	Less to AAC
Weight Comparison	Lighter Weight ~ Possible Lower Transportation Cost	Higher Weight ~ Possible Greater Transportation Cost
Constructability Comparison	Field Modification Possible with Normal Hand Tools	Special Tools Required for Field Adjustments
Material Cost Comparison	Base Material Cost Greater than CMU	Base Material Cost Less than AAC
Insulating Comparison	Alone has Greater R-Value than CMU (R-Value = 11.5)	Alone has Lesser R-Value than AAC (R-Value = 4.5)

In a direct comparison to CMU, AAC is seen as material with a relatively low compressive strength. For low-rise construction, the need for a higher strength material is not a critical factor; however, it appears that AAC could be competitive for buildings with low to moderate strength requirements. Aside from AAC's low compressive strength, it has other important characteristics that other building materials do not have. In chapter 7 it was shown that the amount of reinforcement required for wall in a one story building is comparable to that of CMU. This means that even though AAC has a lower compressive strength, the amount of steel required to resist the same loads is relatively the same. Also, AAC's value as an insulating material is greater than that of CMU. This results in faster construction times and less cost because of the instillation and material cost of additional insulating materials with CMU. AAC's lightweight characteristics has the possibility of increasing the efficiency of the construction process by shortening the time and cost for transportation (higher volume less time) and possibly reducing labor costs (walls can perhaps be constructed faster). To conclude, AAC is a lightweight concrete material that deserves as much consideration as CMU, or even steel or wood, as a building material in the United States.

References

AAC masonry. (2007). *Masonry designer's guide (MDG-5)* (Fifth ed.,)The Masonry Society.

Aercon. *Technical manual*

ASTM International. (2003). ASTM C 1555-03a standard practice for autoclaved aerated concrete masonry. (pp. 1107-1108, 1109)

Autoclaved Aerated Concrete Products Association, Retrieved 5/10, 2007, from http://www.aacpa.org

Autoclaved aerated concrete (AAC). Retrieved 5/10, 2007, from http://www.toolbase.org

The European Autoclaved Aerated Concrete Association, from http://www.eaac.org

Hebel Autoclaved Aeratedd Concrete, http://www.xella-usa.com

The history of AAC. Retrieved 11/9, 2007, from http://www.cfg.co.nz

Kansas Department of Transportation (KDOT) Oversize/Overweight Special Permits, from http://www.ksrevenue.org

Kligner, R. E. *Autoclaved aerated concrete: New possibilities for design.*

Masonry Standard Joint Committee. (2005). Appendix A strength design of autoclaved aerated concrete (AAC) masonry. *2005 masonry standard joint committee code, specification and commentaries*

(ASCE, 2005) *Minimum Design Loads for Buildings and Other Structures,* ASCE/SEI 7-05, Including Supplement No. 1, American Society of Civil Engineers, Reston, VA, 2006

National Concrete Masonry Association. (2005). *Sound transmission class ratings for concrete masonry walls (NCMA TEK 13-1a)*

Pytlik, E. C., & Saxena, J. (1992). *Autoclaved cellular concrete: The building material for the 21st century*A.A. Balkena.

(2006). *RS means building construction cost data* (64th Annual Ed.)

Seismic Hazard Curves and Uniform Hazard Response Spectra, from http://www.usgs.gov

Superior Structures that Save you Time and Money, from http://www.gmchomesfl.com

Tanner, J. E. (2003). Design Provisions for Autoclaved Aerated Cconcrete (AAC) Structural Systems.

Typical thermal properties of common building and insulating materials. (2001). *Priciples of heating, ventilation, and air conditioning* (pp. 5.20-5.23)

Varela, J. L., Tanner, J. E., & Kligner, R. E. (2006). Development of seismic force reduction and displacement amplification factors for autoclaved aerated concrete structures. *Earthquake Spectra, 22*(1), 287-286.

What is Autoclaved Aerated Concrete or AAC?, Retrieved 5/10, 2007, from http://www.aacstructures.com

Appendix A - Design of 10 inch CMU wall

In this section the design of a 10 inch CMU wall for the North wall in the example building will be performed. It will use the same loads for roof dead and snow, as well as the out of plane wind load. Also, in conjunction with the loads being used for the problem the physical properties of the wall section, i.e. the area, moment of inertia, etc, will be taken from the Masonry Designer's Guide 5th Edition (MDG-5).

- Wall construction: 10 inch CMU (Actual dimensions 9.63"x7.63"x15.53")
- Wall height: 12 feet
- $f'_m = 1500\,psi$
- Assume a 48 inch grout spacing
- Use $e = 3.5in$ from North Wall example
- Wall weight: $45\,{}^{plf}\!/_{per\ foot\ of\ wall\ height}$
- Loads:
  - Roof Dead – 47.5 pounds per square foot
  - Roof Snow – 20 pounds per square foot
  - Out of Plane Wind – 20.7 pounds per square foot (outward)

Interpolated from MDG-5:

$$\text{Area} = \left(\frac{40"}{48"}\right)\left(50.4\,{}^{in^2}\!/_{ft}\right) + \left(\frac{8"}{48"}\right)\left(116\,{}^{in^2}\!/_{ft}\right) = 61.33\,{}^{in^2}\!/_{ft}$$

This is to adjust for the use of a grouted cell every 48 inches on center.

$$\text{Width} = \frac{61.33\,{}^{in^2}\!/_{ft}}{12\,{}^{in}\!/_{ft}} = 5.11in$$

$$\text{Moment of Inertia} = \left(\frac{40"}{48"}\right)\left(635\,{}^{in^4}\!/_{ft}\right) + \left(\frac{8"}{48"}\right)\left(892\,{}^{in^4}\!/_{ft}\right) = 677.83\,{}^{in^4}\!/_{ft}$$

$$\text{Radius of Gyration} = \sqrt{\frac{677.83in^4}{61.33in^2}} = 3.32in$$

$$\text{Slenderness Ratio} = \frac{12\,ft\left(12\,{}^{in}\!/_{ft}\right)}{3.32in} = 43.37$$

The 2005 MSJC Code states in 3.3.5.4, “when the slenderness ratio exceeds 30, the factored axial stress shall not exceed $0.05 f'_m$.

Using the governing load combo of $1.2D + 1.6W + 0.5S$ the factored loads become:

$$P_u = 1.2\left[47.5\,psf\,(9\,ft)\right] + 0.5\left[20\,psf\,(9')\right] + 1.2\left[\frac{1}{2}(45 \times 12)\,plf\right]$$

$$P_u = 603\,plf + 324\,plf = 927\,plf$$

$$W = 1.6\left[20.7\,psf\,(1\,ft)\right] = 33.12\,plf_{outward}$$

Axial Strength Calculation

Nominal Axial Strength – check if CMU is adequate without steel

$$\frac{h}{r} = 43.37$$

$$\Rightarrow P_n = 0.80\left[0.80 f'_m A_n\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

$$P_n = 0.80\left[0.80(1500\,psi)\left(61.33\,{}^{in^2}\!/\!{}_{ft}\right)\right]\left[1 - \left(\frac{144in}{140(3.32in)}\right)^2\right] = 53225.65\,plf$$

$$\phi P_n = 0.90(53225.65\,plf) = 47903\,plf \geq P_u = 927\,plf \qquad \text{OK}$$

The 10 inch CMU wall has a more than adequate strength to support the factored axial load without reinforcement.

Flexural Strength Calculated

Factored Moment

$$M_u = 603\,plf\,(3.5in) + \frac{33.12\,plf\,(12\,ft)^2}{8}\left(12\,{}^{in}\!/\!{}_{ft}\right) = 9264.42\,lb \cdot in$$

$$M_{cr} = \frac{f_r I}{\bar{y}} = \frac{63\,psi\left(677.83in^4\right)}{\left(\frac{7.63in}{2}\right)} = 11193.5\,lb \cdot in$$

The factored moment is less than the cracking moment, therefore 2005 MSJC Eq 3-30 is used to calculate the deflection.

$$\delta_u = \frac{5M_u h^2}{48(900f'_m)I}$$

$$\delta_u = \frac{5(9264.42lb\cdot in)(144in)^2}{48(900(1500psi))(677.83in^4)} = 0.022in$$

The moment at mid height can now be calculated.

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e}{2} + P_u\delta_u$$

$$M_u = \frac{33.12plf\,(12ft)^2}{8}\left(12\,{}^{in}\!/_{ft}\right) + 603plf\,\frac{3.5in}{2} + 927plf\,(0.022in) = 8229.6\,{}^{lb\cdot in}\!/_{ft}$$

Check factored axial stress.

$$\frac{P_u}{A_g} \le 0.05f'_m$$

$$\frac{P_u}{A_g} = \frac{927plf}{61.33\,{}^{in^2}\!/_{ft}} = 15.1psi \le 0.05(1500psi) = 75psi \qquad \text{OK}$$

The "*a*" value that is calculated is similar to the calculation for AAC, instead of the value of $0.85f'_{AAC}$, a value of $0.80f'_m$ is used.

$$a = \frac{P_u + A_s f_y}{0.80f'_m b}$$

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

This is extended to a quadratic equation to solve for the value of A_s. The value for M_n is taken as the calculated M_u divided by the factor $\phi = 0.9$.

$$A_s = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$a_{quadratic} = \frac{-f_y^{\,2}}{1.6f'_m b}$$

$$b_{quadratic} = f_y d - \frac{P_u f_y}{0.80f'_m b}$$

$$c_{quadratic} = P_u d - \frac{P_u^{\,2}}{1.6f'_m b} - \frac{M_u}{\phi}$$

$$a_{quadratic}=\frac{-\left(60,000\,psi\right)^2}{1.6\left(1500\,psi\right)\left(12in\right)}=-125000$$

$$b_{quadratic}=60,000\,psi\left(\frac{9.63in}{2}\right)-\frac{927\,plf\left(60,000\,psi\right)}{0.80\left(1500\,psi\right)\left(12in\right)}=285037.5$$

$$c_{quadratic}=927\,plf\left(\frac{9.63in}{2}\right)-\frac{\left(927\,plf\right)^2}{1.6\left(1500\,psi\right)\left(12in\right)}-\frac{8229.6\,{}^{lb\cdot in}\!/\!_{ft}}{0.90}=-4710.333$$

$$A_s=\frac{-285037.5\pm\sqrt{\left(285037.5\right)^2-4\left(-125000\right)\left(-4710.333\right)}}{2\left(-125000\right)}$$

$$A_{s-}=0.017\,{}^{in^2}\!/\!_{ft}$$

A number 4 bar is adequate for a spacing of 48 inches. This provides $0.20in^2/4\,ft=0.05\,in^2/ft$ reinforcement.

Check the nominal moment capacity with # 4 bars every 48 inches:

$$a=\frac{\left(0.2in^2\left(60000\,psi\right)+927\,plf\left(4\,ft\right)\right)}{0.80\left(1500\,psi\right)\left(48in\right)}=0.273in$$

$$M_n=\left(0.2in^2\left(60000\,psi\right)+927\,plf\left(4\,ft\right)\right)\left(\frac{9.63in}{2}-\frac{0.273in}{2}\right)=73489.9lb\cdot in$$

$$\phi M_n=0.9\left(73489.9lb\cdot in\right)=66141lb\cdot in\geq M_u=4\,ft\left(8229.6\,{}^{lb\cdot in}\!/\!_{ft}\right)=32918.4lb\cdot in\Rightarrow O.K.$$

 # 4 bars at 48 inches on center vertical are adequate for combined out of plane bending and axial compression.

In-Plane Bending and Shear Strength Calculated

Check In Plane Bending:

The equations to check in plane bending are the same as the equations used in out of plane bending. Use the load combination $0.9D+1.6W$ for highest bending with lowest axial load.

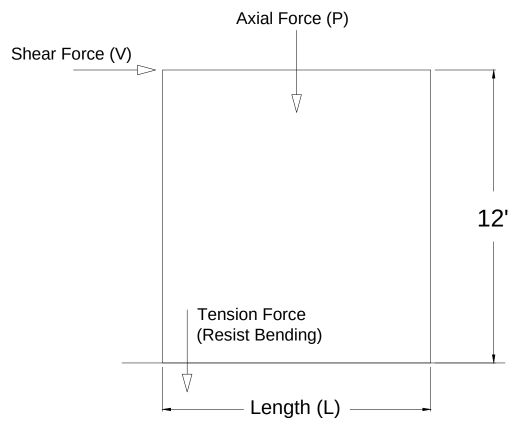


Figure A.1: In Plane Shear on a wall

$$P_u = 0.9(185\,plf + 427.5\,plf) = 551.25\,plf$$

$$M_u = V_w h = 1.6(1733lbs)(12\,ft) = 33273.6lb\cdot ft$$

Check if # 4 bar at $d = 10.67\,ft$ (cell at end of wall) is adequate:

$$a = \frac{\left(0.2in^2(60000\,psi) + 551.25\,plf\,(11\,ft)\right)}{0.80(1500\,psi)(9.63in)} = 1.563in$$

$$d = 10.67\,ft - \frac{551.25\,plf\,(11\,ft)(5.17\,ft)}{0.2in^2(60000\,psi) + 551.25\,plf\,(11\,ft)} = 8.94\,ft$$

$$M_n = \left(0.2in^2(60000\,psi) + 551.25(11\,ft)\right)\left(8.94\,ft\left(12\,{}^{in}/_{ft}\right) - \frac{1.563in}{2}\right) = 1923762lb\cdot in$$

$$\phi M_n = 0.9(1923762lb\cdot in)\left(\frac{1}{12\,{}^{in}/_{ft}}\right) = 144282.15lb\cdot ft \geq M_u = 33273.6lb\cdot ft \Rightarrow O.K.$$

 # 4 bars at ends of wall are adequate for in-plane bending.

Check in-plane Shear Strength of wall

The nominal shear strength of a CMU wall based on masonry and steel has some major differences in calculation. When calculating V_m of a CMU wall the strength is calculated from one equation, equation 3-21 of the 2005 MSJC Code Section 3.3.4.1.2.1. This differs from the calculation of V_{AAC} which determines the minimum of web shear cracking, crushing of the diagonal strut, and sliding shear.

Use the load combo $0.9D + 1.6W$ for highest shear force on wall.

$$P_u = 0.9(185\,plf + 427.5\,plf) = 551.25\,plf$$

Determine in plane shear value V_m

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25P_u \qquad \text{(MSJC Eq. 3-21)}$$

$$\frac{M_u}{V_u d_v} \le 1.0$$

$$\frac{M_u}{V_u d_v} = \frac{33273.6 lb \cdot ft}{1.6(1733 lbs)(10.67\,ft)} = 1.125 \Rightarrow \frac{M_u}{V_u d_v} = 1.0$$

$$V_m = [4.0 - 1.75(1.0)]\left(11\,ft\left(12\,{}^{in}\!/_{ft}\right)(5.11 in)\right)\sqrt{1500\,psi} + 0.25(551.25\,plf)(11\,ft) = 60295 lbs$$

$$\frac{M_u}{V_u d_v} \ge 1.00$$

$$\Rightarrow V_n \le 4A_n\sqrt{f'_m} \qquad \text{(MSJC Eq. 3-20)}$$

$$V_n \le 4\left(11\,ft\left(12\,{}^{in}\!/_{ft}\right)(5.11 in)\right)\sqrt{1500\,psi} = 104496 lbs$$

$$V_m = 60295 lbs = V_n \le 104496 lbs \Rightarrow O.K.$$

$$V_u = 1.6(1733 lbs) = 2772.8 lbs$$

$$\phi V_n = 0.9(60295 lbs) = 54265.5 lbs \ge V_u = 2772.8 lbs$$

Wall is adequate without shear reinforcement to resist in-plane shear forces.